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ELLIS

ON

REINFORCED-CONCRETE WHARVES AND WAREHOUSES AT LOWER POOTUNG, SHANGHAI.

POPPLEWELL

ON THE

STRESSES IN STEEL AND CONCRETE REINFORCED-CONCRETE COLUMNS.

BURR

ON

COMPOSITE COLUMNS OF CONCRETE AND STEEL.

REINFORCED-CONCRETE WHARVES AND WAREHOUSE AT LOWER POOTUNG, SHANGHAI.

BY

SOMERS HOWE ELLIS, M. Inst. C.E.

THE DIRECT EXPERIMENTAL DETERMINATION OF THE STRESSES IN THE STEEL AND IN THE CONCRETE OF REINFORCED-CONCRETE COLUMNS.

BY

WILLIAM CHARLES POPPLEWELL, M.Sc., Assoc. M. Inst. C.E.

COMPOSITE COLUMNS OF CONCRETE AND STEEL.

BY

WILLIAM HUBERT BURR, M. Inst. C.E.

WITH AN ABSTRACT OF THE DISCUSSION UPON THE PAPERS.

EDITED BY

J. H. T. TUDSBERY, D.Sc., M. Inst. C.E.,
SECRETARY.

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THE INSTITUTION OF CIVIL ENGINEERS.

Sect. I.—MINUTES OF PROCEEDINGS.

9 January, 1912.

W. CAWTHORNE UNWIN, B.Sc., LL.D., F.R.S., President, in the Chair.

The President stated that the Council, at their meeting that evening had passed the following resolution, and directed that it be forwarded to the family of the late Sir James Inglis:

> "The Council have learned with great regret the death of their esteemed Past-President, Sir James Charles Inglis, and desire to convey to Lady Inglis and to the other members of his family an expression of deep sympathy with them in their bereavement."

The Council reported that they had recently transferred to the class of

Members.

John Stennitt Bean.
John Richard Blacker.
Arthur Walter Bradley.
Arthur Reginald Cooper.
John Duncan Campbell Couper, M.A. (*Cantab.*)
Frederick Alfred Cutten.
George William Disney.
Frederic William Duckham.
John Reginald Hare Duke.
George Ellson.
Graves William Eves, B.A.I. (*Dubl.*)
Charles Hassard.
Harley Heckford.
George Forster Hickson.
John Theodore Hunt, B.A., B.A.I. (*Dubl.*)
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Moses Kellow.
Herbert Lapworth, D.Sc. (*Birmingham.*)
Arthur Percy Maddocks, B.Sc. (*Birmingham*).
Charles James Mathews.
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James Valence Nimmo, B.Sc. (*Glas.*)
James Harvey Williamson Park, B.Sc. (*Edin.*)
George Stephens Perry.
Fulton Robertson, B.Sc. (*Edin.*)
Alan Stapylton.
Jonas Molesworth Stawell.
James Falshaw Watson.

And had admitted as

Students.

ALEXANDER FARRINGDON CAMPBELL.
EDMUND GRAHAM CLARK, B.Sc. (*Durham*).
HENRY GEORGE DINES.
CLEMENT NIXON ELLIOT, B.Sc. (Engineering) (*Lond.*)
GEORGE FRECHEVILLE.
THOMAS ARCHIBALD MONTGOMERIE HILL, B.A. (*Cantab.*)
THOMAS GIBSON HUSBAND.
JAMES MILLAR KIRKWOOD, B.A. (*Cantab.*)
ROGER CHADWICK LEACH, B.A. (*Cantab.*)
CHARLES BENFIELD POOL, B.Sc. (Engineering) (*Lond.*)
GRAHAM LAUDER WATSON.
ARCHIBALD MUIR WHITE.

The Scrutineers reported that the following Candidates had been duly elected as

Members.

LAWFORD HOWARD FRY.
CHARLES HERBERT GADSBY.
EBENEZER HALL-BROWN.
EDWIN PERCY RICHARDS.
GEORGE DUNCAN SNYDER.
Prof. ERNEST WILSON.

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JOHN GOLDWELL AMBROSE, Stud. Inst. C.E.
ALFRED JAMES BAKER.
MAURICE WILLIAM BRAYSHAY, M.Sc. (*Leeds*).
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HARRY GRAHAME BUTCHART, Stud. Inst. C.E.
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CYRIL STRACHEY HAWDON.
CHARLES FROGGATT HEMINGWAY, Stud. Inst. C.E.
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LIONEL PAGE.
REGINALD SCOTT VICTOR PARSONS.
GEORGE REYNOLDS.
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NEWBOLD CROFTON STAVELEY.
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EMRYS JOHN WHELDON, B.A. (*Cantab.*)
WILFRID EDWARD YOUNG, Stud. Inst. C.E.
AUGUSTUS JULIUS RANELAGH DE ZOYSA.

Associate.

JOHN SYMONDS.

(*Paper No. 3962.*)

"Reinforced-Concrete Wharves and Warehouses at Lower Pootung, Shanghai."

By SOMERS HOWE ELLIS, M. Inst. C.E.

THE works to be described are situated on the Whang Poo River, close to Shanghai, in Northern China. Shanghai itself lies entirely on the north-west bank of the Whang Poo, which is, properly speaking, a tidal creek, about ½ mile wide at this point—15 miles from the River Yangtsze and about 50 miles from open sea. The district of Pootung lies on the opposite side of the river, and consists merely of a line of detached wharves with sheds or warehouses on the strip of bank between them and the cultivated land behind, having no land communication except by native footpaths. All goods have consequently to be lightered to and from the wharves.

The property in question belongs to Messrs. Alfred Holt and Company, of Liverpool, and the wharves and warehouses constructed thereon are for the use of their steamers plying between England and the Far East, Shanghai being an important port of call and of transhipment for the North China coast and the River Yangtsze.

The works consist of, first, a reinforced-concrete skeleton wharf on piles of the same material; secondly, a reinforced-concrete quay-wall alongside a tributary creek; and thirdly, two four-story warehouses constructed in reinforced concrete with brick panel walls. There are in addition a three-story building for offices, staff quarters, etc., three single-story steel sorting-sheds, and finally a power installation of gas-producers, engines, and dynamos, for providing light and power throughout the property. These are all shown in Fig. 1, Plate 1.

The only real difficulty in connection with the works arose from

the nature of the foundations. The whole neighbourhood of Shanghai is founded on river-silt, deposited in past ages by the Yangtsze, and extending to an unknown depth. Borings more than 400 feet deep have been made, which have penetrated nothing but mud of various degrees of solidity, except for layers of impure sand at irregular intervals, a thin intermittent bed of gravel at about 300 feet, and a more regular bed of gravel at a little more than 400 feet from the surface. The top layer of ground, averaging 25 feet in thickness, may usually be reckoned upon to form a solid crust of stiff, sandy clay, though rendered irregular on its surface by the presence of old creek-beds, filled with softer mud. Dependence must therefore be placed on this upper layer of firm ground for bearing superincumbent loads.

The means adopted in the works under consideration were of two kinds; in the case of the wharf, quay-wall, and lighter buildings, the load was borne by groups of piles, supported by skin-friction of the mud on their sides; in the case of the warehouses a raft of reinforced concrete covering the entire area of the foundations carried the weight of the building without assistance from piles.

Wharf.

The wharf, 1,160 feet long on the face and 174 feet wide, has on the river side 21 feet at low water of ordinary spring-tides (Figs. 2, Plate 1). It is assumed that the frontage can be dredged to a slope of 3 to 1 from the face of the wharf down to 24 feet 6 inches, when required.

The piers of the wharf are 15 inches square and spaced 15 feet from centre to centre in each direction: each is supported on a group of four 14-inch piles. Girders run across the wharf from side to side over each row of piers, and support a system of cross beams, spaced 5 feet apart between centres, and a continuous reinforced-concrete deck 5 inches thick. At the front of the wharf two heavy longitudinal beams run from end to end, to carry the rails of a standard-gauge road for a 5-ton crane. The pier-bases (or pile-caps) are connected both longitudinally and transversely by horizontal members, the wharf being further stiffened by a system of diagonal bracing from front to back at each row of piers, and also longitudinally for the last two bays at each end.

Over the greater part of its length the wharf is covered in up to 52 feet from the face, by sheds of steel and corrugated iron, the piers which carry the stanchions for these being borne by a larger number of piles. In line with the front of the shed a training-wall of reinforced-concrete sheet-piles is provided through-

out the length of the wharf, and is continued upwards to above high-water level by a curtain of reinforced-concrete slabs; this acts as a retaining-wall to the silt.

The deck of the wharf is designed to carry a load of 500 lbs. per square foot (so as to bear safely heavy concentrated weights), and the general structure a deck-load of 350 lbs. per square foot. This brings an approximate maximum load of 11 tons on each pile. It was realized from preliminary tests that according to no recognized formula for pile-driving could an apparent factor of safety be obtained, as the set of the pile under each blow of the hammer (provided that driving was continuous) lessened very little after an average penetration of 15 feet, and sometimes tended to increase after a penetration of 25 feet. The Author was convinced, however, that the supporting power of the mud was comparatively high when time was given for the film of water between it and the pile to escape and conditions of true skin-friction to obtain; and an arbitrary figure of 5 cwt. per superficial foot of the pile, exclusive of its own weight, was fixed as its maximum bearing-capacity.

The piles were driven by a steam pile-driver, which gave a rapid succession of blows with the falling weight of a 3½-ton hammer dropping 6 to 8 inches. The weight of the cylinder, etc., rested on the pile in the meantime, with a steady weight of 1½ ton. A final set of 1 inch per blow was usually obtained, and no anxiety was felt as to the future stability of the majority of the piles. In some places, however, much softer mud was encountered. As an extreme case, in one pier the piles sank rapidly under the steady weight of the hammer and cylinder, without any driving, and had to be checked by holding up this dead weight when they were at the required depth. This was naturally felt to be unsatisfactory, and after a lapse of 1 month the four piles of the group in question were tested with the maximum load of 45 tons, under which they settled 1¾ inch in a fortnight, coming to rest at the end of that time. They were afterwards lengthened 8 feet and driven further. For the first eighty blows of the hammer, at the start of this operation, the pile struck did not move. After this, when the adhesion between the mud and the pile was broken, it went down with ease, though not actually without driving, as it had done before. A subsequent test on the group of piles showed a total settlement of 1½ inch under the load of 45 tons, so that it was decided not to be worth while to lengthen and redrive any more piles. This group showed results worse than any other, both while driving and under the test-load; and as the

actual conditions of loading, when the wharf was finished and braced together, would be much less severe on any one pier, there was not felt to be much cause for anxiety.

The most favourable anticipations have so far been realized by the behaviour of the wharf in practice, when it has been subjected to the test of having considerably more than the estimated maximum load stacked upon it locally, without any appreciable settlement or sign of distress in the superstructure.

The piles shown in Figs. 6 were moulded in a horizontal position, and a period of 2 months was specified for maturing, but in warm weather some were driven 6 weeks after being moulded, without ill effect.

No iron or steel helmet was used for driving. A thin piece of pine board was placed directly on the head of the pile; on this a $\frac{3}{8}$-inch steel plate, and above that again a piece of hard wood, 4 inches thick. An ordinary short timber dolly received the actual blows of the hammer. Great care was taken to keep both the pile and the guides of the driver in an absolutely vertical position, so as to strike each blow squarely on the pile-head. The pile ran in the guides between rubber-covered rollers packed with thin pine battens which travelled downward with it, and prevented any lateral movement. The heads of the piles were in most cases unbroken by driving; and in no instance did the damage exceed a chipping off of the top corners.

With the exception of the pile-caps, beams, and deck, all the wharf-members were moulded on shore, thereby avoiding for the most part the unsatisfactory operation of concreting below high-water level. For erection, a timber platform was clamped firmly on to the heads of each group of piles at the level of the base of the caps, allowing the heads of the piles to project through, from which the concrete was then cut away, leaving the steel rods bare. The horizontal bracing running parallel to the length of the wharf was then laid in position, the ends of these longitudinals resting (on packing-pieces) on the timber platforms clamped to the pile-heads. The longitudinal members then served as a support for the whole of the temporary staging from which the rest of the superstructure was erected.

The steel rods projecting from the ends of the various horizontal, diagonal and vertical members overlapped or were hooked round each other within the area of the pile-caps, which were then concreted in situ, forming a rigid connection to the piles, bracing, and columns. This concrete was run into place down inclined shutes at low tides, and was well worked among the reinforcement with steel

rods. At neap-tides on the Whang Poo River the ebb lasts for about 8 hours, and low water for 3 or 4 hours, so that there was ample time to concrete six to eight caps during a daylight tide; and, when covered with mats, the partially-set concrete suffered no damage from the next flood.

When a quarter of the length of the wharf was so far completed in this way, wooden moulds for the girders, beams, and deck were set over the entire area of one-eighth the length of the wharf—about 26,000 square feet. The reinforcement was then placed in position for successive strips two bays (30 feet) in width across the wharf, and concreting was carried on so as to complete this section of the structure entirely in a continuous series of working-days. The surface of the deck was sloped downwards towards the centre of each square 15-foot bay, where a drainage-hole and grating were provided. The wharf was completed for use by the provision of a system of creosoted Oregon-pine fender-piles, walings, and caps along the face, bolted into the concrete at deck and lower-boom level.

The moulding of piles for the wharf was commenced on the 16th January, 1908: driving was begun on the 2nd April and completed on the 31st December of the same year, 4,023 piles having been driven in that period. Some delay was then caused by the driving of sheet-piles for the training-wall, and the first pile-caps were not concreted until the 20th March, 1909. Concreting of the deck was commenced on the 10th June and completed on the 20th July, 1910, a period of $2\frac{3}{4}$ years being thus occupied in the whole construction. The fitting of fenders, bollards, etc., was finished 3 months later, and the erection of the sheds about the same time.

The climate of Shanghai is marked by extremes of temperature, from tropical heat in summer to frequent sharp frosts in winter.

The part of the wharf-deck which was concreted during the summer of 1909 developed a series of hair cracks along the centre of each alternate bay (i.e., 30 feet apart) across its full width, during the succeeding winter; the positions of the cracks coinciding roughly, though not exactly, with the places where work had ceased for the day. Many of these cracks went through to the underside of the deck, and water was observed to ooze through them after rain. They were subsequently cut out, $1\frac{1}{2}$ inch deep, and a straight open joint was formed in the concrete surface which was run with hot asphalt. The reinforcement, where exposed by this operation, was nowhere corroded; indeed, at the end of 1911 no signs of rust-stains were visible anywhere on the superstructure or pile-heads of the wharf.

Quay-Wall.

The bank of a tributary creek forms the eastern boundary of the property, and berthage-space for lighters was required here.

The present wall, 495 feet in length, was constructed of concrete throughout (Figs. 7 and 9, Plate 2). It is founded on cross rows of three 12-inch square piles, of which the front row serve as king-piles to a continuous line of sheet-piles (Figs. 6, Plate 2), which forms the face of the wall up to a height of 12 feet below quay-level. The heads of the piles are connected by a deck 14 feet wide, above which the wall consists of a nearly vertical slab tied back to the deck by a series of triangular counterforts.

Stability against overturning is secured by the weight of the filling—dry earth, rammed in layers—on the deck, plus the anchoring hold of the back piles, it being assumed in the calculations that the cross-sectional area of the earth and concrete within the limits of the piles and counterforts may be taken as one solid mass, with their combined weights acting together in one vertical plane determined by their several positions.

The original design for the wall showed a series of anchor-ties from the coping back to the foundation-piles of the shed immediately behind it. Before these were put in, however, it was necessary to back up the wall, and for this purpose earth was removed from the front down to low-water level. Just as this operation was completed, a slight forward movement of the wall was detected, amounting to a maximum of $2\frac{1}{2}$ inches at coping level at the centre of its length, and apparently extending downwards to the piles without much diminution. The earth backing was at once taken out and dumped again in front of the wall, which went back in the course of a few weeks to within $\frac{1}{2}$ inch of its original line. A crack in the ground had also appeared, about 40 feet behind the coping-line; and as it was evident that the whole stratum behind the wall was in an unstable condition, it was decided to put in anchorage-ties, attached to the wall at deck level, instead of coping level, and extending back to the foundation-slab of the warehouse, where they were held by the ends of the foundation-beams. These ties are formed of four steel rods, $1\frac{3}{8}$ inch in diameter, overlapping 2 feet 6 inches at the joints, bound with wire ties and struts, and encased in concrete to a finished size of about 15 inches square. The attachments at either end are made by the hooked ends of the rods fitting round links of $1\frac{5}{8}$-inch bars, securely fastened into the wall and the warehouse foundation. A flexible joint at these places is secured by encasing

the steel in a 1-to-1 mixture of asphalt and sand, instead of the usual concrete protection.

Before replacing the backing to the wall a continuous rubble drain was formed behind the back of the deck, and no further movement of the wall has been detected.

After completion of the reinforced-concrete work, creosoted Oregon-pine fender-piles and walings were bolted to the face opposite each counterfort. Cast-iron mooring-posts in concrete blocks were also provided about 60 feet apart.

Warehouses.

The two warehouses (Fig. 1, Plate 1, and Figs. 8, Plate 2), have four stories and a flat roof, each floor being designed to carry a superficial load of 500 lbs. per square foot throughout its area. The entire structure is of reinforced-concrete except the panel walls, which are of brickwork 15 inches thick. The roofs have an external waterproof covering.

Two pairs of electric lifts are installed on the outside of each warehouse. In the alley-way between the two, a system of stairways is provided up to each floor, in view of the prevailing Shanghai practice of carrying goods by means of coolies up to any height or to any distance. The stairways lead to cantilever landings which afford access to both warehouses at the one level. The effect is that of bridges at the head of each stair; but, in order to allow for unequal settlement of the warehouses, the landings are cut at the centre and supported on brackets, which are a continuation of the floor beams on either side.

The chief point for remark in the design of the warehouses is the foundations, wherein special measures had to be taken on account of the heavy load and the yielding nature of the ground. Assuming the floors to be all fully loaded—an extremely unlikely contingency, but one which was reckoned on in calculating the maximum loads—the interior columns, spaced 20 feet from centre to centre in each direction, would bring a load of about 400 tons on to the base of each. Considering the small load which piles can bear safely in this ground, their provision in sufficient numbers to be of much service would have involved unjustifiable expense. Under favourable circumstances the ground around Shanghai will bear a maximum load of 1 ton per square foot without undue settlement; and weighting tests, carried out on the site of the warehouses, tended to show the correctness of this estimate locally. The expedient adopted in order that this figure should not be exceeded was

to form one foundation-slab extending over the whole area of each building, and some distance (actually 7 feet from the column-centres) outside it, reinforcing the slab to such an extent that the space between the various points of bearing should be capable of resisting the upward thrust of the ground; also that in the event of any isolated tendency to excessive settlement, the load should be distributed between any three adjacent bays to an extent sufficient to prevent cracking of the concrete. This raft is shown in Figs. 8 and 10, Plate 2.

In the neighbourhood of Shanghai settlement occurs in all structures of any weight, and the buildings under consideration are no exception. Taking the pair of warehouses together, they form one block 314 feet by 239 feet in area, with their foundation-slabs so near together as to act as one in affecting the stratum of ground at the depth (probably up to 50 feet) where compression takes place. The total settlement during the last 6 months of construction and the year after completion of the buildings has amounted to about 14 inches at the two centre points of the area (half way along each warehouse on their inner sides), and 10 to 12 inches at the four outside corners: showing clearly the wave-like subsidence of the strata compressed.

Owing probably to the strength and elasticity of the foundation slab, no cracks or other sign of distress have appeared anywhere in the concrete structure. The inward tilt of the buildings has not exceeded 1 inch in a height of 64 feet. It is probable that settlement, though now nearly stopped, may continue for some years; but it is not anticipated that practical inconvenience will be caused thereby.

One curious feature of the movement is that the buildings have closed bodily together, to the extent of about 1 inch, probably owing to alternate expansion and contraction on a base not now truly horizontal.

Office Building.

Near the warehouse is the office building (Fig. 1, Plate 1), a three-storied structure of similar character to the warehouses but on a lighter scale. It is carried on piling, a maximum load of 11 tons per pile, as in the wharf, being allowed. There is, in addition, the bearing-capacity of the concrete slab, which is a more indeterminate quantity.

In contrast with the behaviour of the wharf, some subsidence of this building has taken place, amounting to about 3 inches in all, and a slight irregularity of settlement of the wave-like character already referred to, though not sufficiently marked to cause anxiety.

Concrete.

The concrete throughout is composed of 1 part Portland cement, 2 parts sand, and 4 parts beach-gravel, except for the foundations of the buildings, where the proportions are 1 : $2\frac{1}{2}$: 5.

The cement was obtained chiefly from Heidelberg, about 2,000 tons only being supplied from England in the early stages of the work. In all cases it was tested on the site for tensile strength, etc., according to the British Standard specification, to which all consignments were found to conform without aeration.

A number of compression tests were also carried out on 8-inch cubes of concrete taken from the mixer on gauging, so as to obtain a periodical record of the material actually used in the work. The following results were obtained from three series of successive tests on concrete of 1 : 2 : 4 mixture :—

Series.	Number of Samples.	Average Crushing-Strength in Lbs. per Square Inch.				
		28 Days.	56 Days.	84 Days.	168 Days.	1 Year.
1	45	2,165 (26 tests)	2,581	2,811	3,070 (19 tests)	..
2	56	1,923	..	2,540	3,333	..
3	32	2,251	..	..	..	4,137

In the second series the cubes were kept covered with damp sand, in the others they were exposed to the air (in the shade).

The gravel was screened twice, so as to exclude all material retained on a $\frac{3}{4}$-inch sieve or passing through a $\frac{1}{8}$-inch sieve. The large stones were afterwards broken by hand, and mixed with the selected material. Fine material obtained from the screenings of the gravel was mixed with the sand in the ratio of 1 to 2, as this mixture was found to give a higher compressive strength than when the normal sand was used alone.

Two mechanical mixers of a German type, installed side by side, coped with the whole work. Each charge contained a weighed portion of cement and corresponding measured volumes of sand and gravel (the actual proportions for 1 : 2 : 4 concrete being 188 lbs. of cement, 3·76 cubic feet of sand, 7·52 cubic feet of gravel), so as to make up the contents of a Decauville skip, in which it was afterwards conveyed to all parts of the site. Revolving arms in the barrel of the machine stirred the materials, at first dry, and then with the admixture of sufficient water (varying with the temperature) to

produce a concrete which would just flow when tipped on to the wooden platforms close to the work. It was conveyed from these by hand in rectangular buckets. All labour was performed by Chinese coolies, under the supervision of German foremen on the part of the contractor, and English inspectors on that of the engineer. When trained to the work the Chinese became thoroughly competent, and the concrete was, as a whole, of a highly satisfactory appearance when stripped of the moulds.

Reinforcement.

The greater part of the steel in the reinforcement is in the form of plain round bars and wire ties. Expanded metal, of $\frac{1}{4}$ inch by $\frac{3}{16}$ inch section and with a mesh 3 inches across, is used for all floors of buildings, and for the deck of the wharf; and a certain proportion of angle-bars, flat-bars and Kahn bars in the wharf-girders and beams, as shown in Figs. 4–6, Plate 1, and Figs. 9–14, Plate 2.

The steel was specified to be capable of bearing a tensile stress of between 60,000 lbs. and 70,000 lbs. per square inch, with an elongation of not less than 20 per cent. in a length of 8 inches. In making the calculations[1] a working-stress of 16,000 lbs. per square inch was allowed in tension, and in compression fifteen times the stress on the concrete with which it was in contact, up to a maximum of 12,000 lbs. per square inch. A maximum stress of 530 lbs. per square inch in compression and 55 lbs. per square inch in shear was allowed on the concrete of girders and beams, and a compressive stress of 450 lbs. per square inch on the concrete of columns with plain-tie reinforcement, taking into account the whole cross-sectional area. In the case of the hooped warehouse-columns shown in Fig. 13, Plate 1, a maximum stress of 1,200 lbs. per square inch on the concrete within the hooping is calculated upon, neglecting that outside. It was important to keep the size of these columns within moderate limits; and, in view of the heavy load on those from the ground to the first floors, the only alternatives were to have steel stanchions encased in concrete, or to adopt this system of hooped reinforcement, with heavily stressed concrete confined within it. As described hereafter, five experimental columns, 10 inches in diameter, were made in accordance with this design, and tested to destruction before the warehouse-columns were actually commenced;

[1] The methods of working out the details of design are those given in Messrs. Marsh and Dunn's work, "Reinforced Concrete."—S. H. E.

and it was apparent from these tests, as from those made elsewhere, that a sufficient factor of safety was obtained.

The interior first-floor columns (Figs. 13) are designed to carry a maximum load of 376 tons. They are octagonal in section, and the $2\frac{1}{4}$-inch vertical bars are kept in place by a series of inner circular bands to which the vertical rods are fastened by means of U bolts. The outer hooping is a continuous rod $\frac{1}{2}$ inch in diameter, wound spirally with a pitch of $2\frac{3}{4}$ inches. The operation of winding was effected by fixing the skeleton of main bars (bolted on to the spacer bands) in a horizontal position, free to revolve about its axis, with the end of the hooping fastened to it at one extremity. It was then turned by hand-bars inserted in the skeleton, while the $\frac{1}{2}$-inch rod was fed forward through a check which left the requisite tightness on the finished hooping.

The second and third floor interior columns are similar in design, but of lighter section and reinforcement. The roof-columns are of ordinary rectangular section, with wire ties 6 inches apart.

All exterior columns above the first floor are reinforced in the latter method, continuous spiral winding, with its accompanying high unit stress on the concrete confined within it, being only necessary here below this level. The interior columns of the first three floors are protected from accidental injury by being encased in $\frac{1}{8}$-inch steel plates up to a height of 6 feet. The roof-columns are fitted with angle-bars at the corners for the same purpose.

In calculating the strength of beams and girders partial continuity was assumed, the positive bending-moment for inner spans being taken as $W l/12$, and that for outer spans as $W l/10$, compared with $W l/8$ for freely supported beams. Care was taken to reinforce amply those parts of the structure in which a negative bending-moment due to continuity might involve high stresses.

All steel was covered with $1\frac{1}{2}$ inch of concrete, except in the case of floors, where this dimension was reduced to 1 inch. In structural members exposed to the weather a cover of 2 inches was given in almost all instances. The absence of rust-marks on the concrete surface of the wharf, and the fact that wherever bared the steel is clean, seem to show that this coating of good dense concrete is an entire preventive of corrosion, even when the structure is alternately wet and dry. The Whang Poo River at Shanghai, however, is of fresh water, not salt, though under tidal influence.

Especial care was taken, both in designing the systems of reinforcement, and in preparing the steel skeletons, to ensure that all steelwork was rigidly fastened together, and in its proper position with reference to the moulds, before concreting was com-

menced, with no loose parts to be omitted by accident, or knocked out of place during the process of ramming the concrete. With this end in view, the wharf-girders, which run across the structure every 15 feet, are reinforced with what is in effect a pair of light skeleton girders, built up of $2\frac{1}{2}$-inch angle-bars at the top, and of $2\frac{1}{2}$ by $\frac{5}{8}$-inch flat bars at the bottom, shear members being riveted to them, as shown in Figs. 6, Plate 1. The additional steel required in tension is supplied by two $1\frac{1}{4}$-inch round bars, one turned up over the points of support, and the other bent down to reinforce the concrete in compression at the same points.

For convenience in working, the contractors assembled and riveted together on shore the whole 168-foot length of each of these framed skeletons, afterwards carrying them bodily by a line of coolies into place on the wharf. Under other conditions of labour, it would be easier to build them in short sections, riveting up the cover-plate joints (which are spaced at points of minimum stress) after erection in situ.

This system of reinforcement, though not so suitable for heavily loaded beams of limited depth, as in the warehouses, was found to be convenient and economical for the wharf-girders; and, being self-supporting across the full spans, it might be useful in positions of less accessibility, as the moulds could, if necessary, be hung from the reinforcement.

A corresponding system of rigidly-fixed shear members was obtained in the wharf deck-beams by the adoption of double Kahn bars, 4·8 lbs. per lineal foot, in conjunction with a single round bar at the top and bottom (Figs. 4, 5, and 6, Plate 1). These beams being shallow and lightly loaded, Kahn bars were suitable and practically convenient, though somewhat costly—the contract price being 2·49*d.* per pound, as compared with 1·56*d.* for round bars and ties—owing to the cheapness of the labour which was saved by their use.

In the more heavily loaded beams and girders of the warehouses, therefore, nothing but plain round bars and wire ties were used, as shown in Figs. 10–12, Plate 2. The greater part of the shearing stress was taken up by the tension-bars, which are bent up at intervals to pass along the top of the member over the points of support. To resist the remaining shear, to confine the concrete, and also to link together the main reinforcement, three rows of stirrups were employed. These are made of two wires, either $\frac{1}{4}$ inch or $\frac{5}{16}$ inch in diameter, twisted together, with their free ends clenched tightly round the longitudinal bars at top and bottom. The sets of bars are tied together horizontally with wire links, and

the whole skeleton is tightened and stiffened by struts $\frac{1}{2}$ inch in diameter, at intervals of 3 to 5 feet, the final introduction of which converts the assemblage of bars and ties into a rigid framework.

Testing.

It was not thought advisable to subject any part of the finished structures to test loads greatly in excess of the maximum safe load allowed for. At an early stage of the work, however, tests to destruction were carried out on one experimental wharf-beam of full size, including a deck-slab 5 feet wide and reinforced according to the final design (Figs. 5 and 6, Plate 1); also on one warehouse-beam of similar character (Fig. 11, Plate 2), and on five columns designed to represent to a reduced scale the lower warehouse-columns (Figs. 14, Plate 2).

The beams were both made of such a length as to project half a span on each side of the supports, and were loaded in such a way that, when the full (safe) load came on the (centre) span to be tested, the cantilever ends were weighted to a sufficient extent to give the effect of a continuous beam with one span loaded and the two adjacent spans light. This outlying weight, however, was not increased with the subsequent loading to destruction.

Wharf-Beam.—For example, the wharf-beam (Figs. 4 and 6, cross section B B), which was calculated to bear a distributed load of 1,750 lbs. per lineal foot, was loaded with a maximum of 400 lbs. per lineal foot on the projecting arms.

This beam measured 14 inches from the top of the deck to the centre of the reinforcement; it was 11 inches wide, and carried a deck-slab 5 feet wide by 5 inches thick. The results of the test were as follows:—

Wharf-Beam, Age 3 Months, Span 15 Feet.

Date, 1908.	Super-Load.	Total Load.	Deflection.	Remarks.
	Tons.	Tons.	Inches.	
August 8 .	11·7	14·6	0·062	Calculated super-load.
„ 11 .	11·7	14·6	0·081	After 3 days.
„ 17 .	22·8	25·7	0·112	
„ „ .	34·0	37·0	0·162	
„ 18 .	34·0	37·0	0·174	After 24 hours.
„ „ .	45·0	48·0	0·310	First crack appeared (in underside).
„ 20 .	90·0*	93·0	..	Beam suddenly collapsed, as described overleaf.

* $7\frac{2}{3}$ times calculated super-load.

The beam tested on the 20th August first began to fail by diagonal tension near the supports; the concrete then stripped from the underside of the tension-bars, which drew, fracturing the diagonals of the Kahn bars at their points of junction. The concrete then crushed at the inner edges of supports (as the beam sagged), and finally at the surface of the deck-slab, while cracks opened in the beam below it.

Warehouse-Beam (Figs. 11, Plate 2).—In testing the warehouse-beam sufficient precautions were not taken to prevent settlement of the points of support, which subsided unequally. The deflection readings were, in consequence, not accurate enough to be of value. The beam failed under a distributed load of 169 tons—about $7\frac{1}{2}$ times the calculated safe load—on a span of 20 feet.

Experimental Columns.—These were tested at 15 months. They were octagonal in shape, 10 inches across, 6 feet 3 inches in length. The reinforcement consisted of eight rods $\frac{3}{4}$ inch in diameter, with $\frac{3}{16}$ inch helical hooping at 1 inch pitch, enclosing an area 8 inches

Concrete.	Load when First Crack appeared.	Final Load.	Calculated Safe Load.
	Tons.	Tons.	
1 : $1\frac{1}{2}$: 3	132·2	170·4	Area of steel in compression 3·53 square inches.
	132·2	171·8	
Average	132·2	171·1	Area of concrete inside hooping 50·27 − 3·53 = 46·74 square inches. Safe load in lbs. = 3·53 × 12,000 + 46·74 × 1,200 = 44 tons.
1 · 2 : 4	139·1	155·8	
	132·2	172·5	
	146·1	167·0	
Average	139·1	165·1	

in diameter (Figs. 14, Plate 2). These columns all failed by bending, although the length was only nine times the diameter of the area confined by the hooping. Actual failure did not take place until considerably after the maximum loads were recorded. A slight bending at this stage, accompanied by cracks in the surface concrete at one end, reduced the pressure indicated on the (hydraulic) testing-machine gauge. As further pressure was applied, the column bent increasingly, gradually scaling off the outer coating of concrete, and finally breaking a strand of the wire hooping at the point of greatest tension, while the inner core of concrete was to all appearances still intact.

By way of comparison with these reinforced columns, some of a similar section, but unreinforced, were made and broken at the same time, with the following results.

Concrete. Age 15 Months.	Load applied when First Crack appeared.	Final Load.	Equivalent Load on Full Area.
	Tons.		Lbs. per Square Inch.
1 : 1½ : 3	90·4	111·3	3,040
	97·4	107·8	2,945
	130·8	130·8	3,573
Average	106·2	116·6	3,186
1 : 2 : 4	86·3	86·3	2,357
	100·2	106·4	2,906
	76·5	80·7	2,204
	97·4	109·9	3,002
Average	90·1	95·8	2,617

These all crushed in the normal manner, with the formation of a conical fracture at one end.

In addition to the tests to destruction, test working-loads (about 72 per cent. of the maximum calculated safe load) were imposed on four ground-floor columns of the warehouse first constructed, before completion of the building. In order to effect this, 300 tons of steel bars were piled on the first and second floors around the column to be tested—bringing about 270 tons load upon its ground-to-first-floor length. In the ordinary working of the warehouses this is as much as can be brought to bear upon them. This loading brought on to the floor-beams and girders concerned a load about 25 per cent. in excess of their maximum calculated load. No sign of weakness or distress was perceptible anywhere, nor was there any observable compression of the columns.

Equipment.

Owing to the prevailing practice in Shanghai of carrying all loads up to 4 cwt. on bamboos between two coolies, the only equipment provided on the wharf consists of two 5-ton steam cranes, to travel on a standard-gauge track along the front and beside the creek. These are for the purpose of lifting into lighters weights of more than 1 ton, which cannot easily be handled by coolies. The vessels of Messrs. Holt's " Blue Funnel " fleet are exceptionally well provided with gear, discharging their cargo as fast as it can be coped with on shore.

The power installation consists of a suction-gas producer-plant, two gas-engines of 150 B.HP. each, linked with dynamos, and a third of 25 B.HP. for night use when no cargo is being discharged.

Overhead cables convey the current to a main switchboard in the office, and thence to the warehouse lift-motors, and electric lights throughout the site.

Costs.

Owing to the wide fluctuation in exchange, there is a difficulty in stating the costs of the work exactly in English currency. When the contract was let the value of the Shanghai tael was approximately 3*s*., but soon afterwards it fell rapidly to a minimum of 2*s*. 4½*d*. Taking into account the material ordered from Europe at varying exchanges, and the practically uniform rates for labour, a value of 2*s*. 8*d*. per tael represents fairly the actual cost of the work. On this basis the unit costs of the chief items are:—

Wharf.

Item.	Unit Cost.	Unit.	Percentage of Total Cost of Structure.
	s. *d.*		
Bearing piles	3 7·4	Cubic foot	40·2
Sheet-piles in training-wall . . .	4 5·4	,,	6·5
Superstructure	3 5·1	,,	51·0
Timber fenders	3 1·3	,,	2·3
			100·0

Amounting to 8*s*. 7*d*. per square foot of deck area.

Quay-Wall.

Item.	Unit Cost.	Unit.	Percentage of Total Cost of Structure.
	s. *d.*		
Excavation (including front of wall) .	6·4 to 1 7·2	Cubic yard	13·0
Filling	6·4	,,	
Bearing and king-piles	4 4·0	Cubic foot	19·7
Sheet-piles	4 3·8	,,	23·3
Superstructure	2 2·0	,,	25·6
Anchor-ties (including excavation and attachments)	10 0	Lineal foot	12·9
Timber fender	3 1·3	Cubic foot	6·5
			100·0

Amounting to £13 11*s*. 7*d*. per lineal foot of wall.

WAREHOUSES.

Item.	Unit Cost.	Unit.	Percentage of Total Cost of Structure.
	s. *d.*		
Excavation	4·8	Cubic yard	3·6
Lime concrete	10 8·0	,,	
Reinforced-concrete foundations and ground floor	2 3·4	Cubic foot	24·4
Reinforced-concrete superstructure	3 0	,,	55·7
Brickwork in walls	1 8·0	,,	6·7
Waterproof roofing	5·3	Square foot	1·6
Doors, windows and steelwork in roof over alley-way	..	..	8·0
			100·0

Amounting to 5·17*d.* per cubic foot of content, exclusive of lifts and electric light installation.

The cost of unskilled labour in Shanghai varies from 10*d.* to 1*s.* per day.

The Contractors for the work were Messrs. C. Vering, of Hamburg, who carried it through without the intervention of sub-contractors, except for brickwork and excavation, but were not in any way responsible for the design.

The Author was Engineer, in company with Messrs. Davies and Thomas, Architects, of Shanghai. The Resident Engineer was Mr. Guy Magee, who was also largely concerned in getting out the details of design. Mr. Magee's health having broken down towards the close of the work, he was succeeded in charge by Mr. A. J. Wynne, formerly his chief assistant.

The Author desires to express his appreciation of the ungrudging services rendered by these gentlemen, and by the other members of the staff.

The Paper is accompanied by five tracings, from which Plates 1 and 2 have been prepared; also by a number of photographs.

(*Paper No. 3953.*)

"The Direct Experimental Determination of the Stresses in the Steel and in the Concrete of Reinforced-Concrete Columns."

By William Charles Popplewell, M.Sc., Assoc. M. Inst. C.E.

In the best designs for concrete columns reinforced by longitudinal steel bars it is generally assumed that any shortening caused by the application of the load is the same in amount for the steel as for the concrete; that is to say, that the two behave as one material without any sliding of the one upon the other. In the simple columns reinforced by four round steel bars, used for the present experiments, the presence or otherwise of lateral tie-bars does not affect the calculations. Further, the load carried by the column was supposed to be applied axially, and the column was of such a length that the influence of bending, such as takes place in what are termed "long columns," was non-existent or negligible.

Preliminary Considerations.

Let E_c denote the modulus of elasticity of the concrete;
E_s ,, the modulus of elasticity of the reinforcing steel;
A ,, that part of the cross section which consists of concrete;
a ,, the cross section of the whole of the steel;
C ,, the intensity of the compressive stress in the concrete;
S ,, the intensity of the compressive stress in the steel;
W_c ,, that part of the load borne by the concrete;
W_s ,, that part of the load borne by the steel;
W ,, the whole load.

The application of the load causes a shortening, x, of some length, l, of the column, which is bounded by two parallel planes normal to the axis. Upon such assumption the shortening is the same for the steel as for the concrete. That is to say :—

$$x = \mathrm{C}\frac{l}{\mathrm{E}_c}, \text{ or } x = \mathrm{S}\frac{l}{\mathrm{E}_s};$$

therefore

$$\mathrm{C} = \mathrm{S}\frac{\mathrm{E}_c}{\mathrm{E}_s}.$$

But

$$\mathrm{C} = \frac{\mathrm{W}_c}{\mathrm{A}} \text{ and } \mathrm{S} = \frac{\mathrm{W}_s}{a} = \frac{\mathrm{W} - \mathrm{W}_c}{a},$$

so that

$$\frac{\mathrm{W}_c}{\mathrm{A}} = \frac{(\mathrm{W} - \mathrm{W}_c)}{a}\frac{\mathrm{E}_c}{\mathrm{E}_s} = \frac{\mathrm{W} - \mathrm{W}_c}{a\,m};$$

where

$$m = \frac{\mathrm{E}_s}{\mathrm{E}_c}.$$

Therefore

$$\mathrm{W}_c = \frac{\mathrm{A\,W}}{a\,m + \mathrm{A}}, \text{ and } \mathrm{W}_s = \mathrm{W} - \mathrm{W}_c,$$

or

$$\mathrm{C} = \frac{\mathrm{W}}{a\,m + \mathrm{A}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (1)$$

$$\mathrm{S} = \frac{\mathrm{W}\,m}{a\,m + \mathrm{A}} \quad . \quad . \quad . \quad . \quad . \quad . \quad (2)$$

Thus, knowing the total load carried by a column, its outside dimensions, the size of the reinforcing bars, and the ratio of the moduli of elasticity of the materials employed, it becomes easy to calculate the load borne by the concrete, the load borne by the steel, and the respective compressive stresses in these materials. The accuracy of the calculation depends upon two assumptions, the first of which is that the modulus of elasticity of either material does not alter appreciably during the application of the working-loads. It is known from experimental work already carried out that, within the working-stresses usually employed, there is not sufficient change in the modulus of the concrete to affect materially the accuracy of the calculation, and that the modulus for the steel remains constant under stresses higher than those likely to be induced by the working-loads. The second assumption, that on any length of the column bounded by a pair of parallel planes normal to the axis the shortening of the concrete under any given load is

equal to that of the steel, has already been mentioned. Under certain conditions of loading and construction this latter condition can be guaranteed. Thus, if the reinforcing-bars were carried through the column so as to be exposed at each end, and were cut off flush with the end of the concrete; and if, at the same time, the ends of the column were finished off with perfectly plane surfaces, and the load was transmitted through rigid platens; then, whatever shortening the concrete sustained, the steel would perforce be shortened to the same extent, and there could not possibly be any general movement of the concrete along the steel.

These ideal conditions can never obtain in actual construction, and the load is always applied primarily through concrete, the load in the steel being taken up partly by the pressure of the concrete on the ends of the bars and partly by the grip of the concrete on the steel. During the process of setting, the concrete contracts, and in so doing obtains a hold on the bars; this is often called adhesion, but is more probably in the nature of a frictional grip preventing any relative motion of the surfaces of the steel and the concrete.

Without attempting to reveal the exact nature of what takes place during the gripping of plain steel bars by the shrinkage of the concrete, the present experiments were undertaken with the object of finding out, if possible, whether there is any movement of the steel in the concrete under ordinary conditions of working. In view of the fact that this point has often been the subject of controversy, and of the importance of having some more definite knowledge than is available at present, the Author thought it worth while to make some attempt towards an experimental solution.

In order to make such determination it became necessary to devise some means whereby the strain both in the reinforcement and in the adjoining concrete could be measured independently, at the same time and under the same load.

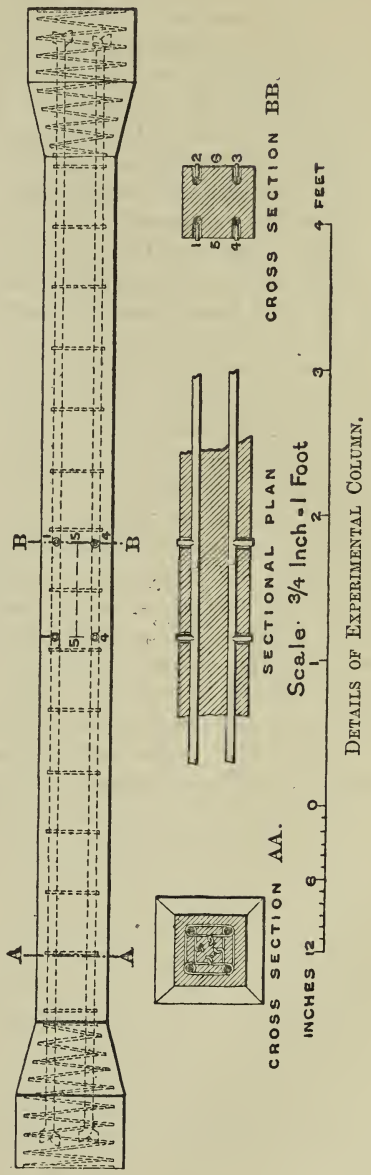
In order to do this it was necessary that the observer should be able to take simultaneous readings on an extensometer attached to the steel and on a second one fixed to the concrete in the neighbourhood of the reinforcement. On consideration the Author decided that the only feasible way of measuring the strain in the steel would be to ascertain the relative movement of two pins, attached to the reinforcement and projecting to the surface of the concrete.

This principle was applied in a small trial column on which a few experiments were carried out in order to find out how far the scheme of measurement was practicable, and also whether the information provided was likely to be reliable. The pillar in question was 4 inches square in section and had a single $\frac{1}{2}$-inch steel bar

running down the centre from end to end. Between each end of the pillar and the platen of the testing-machine was a layer of neat cement. Forged on to the main bar and at right angles to it were four short pins which projected about $\frac{1}{16}$ inch beyond the surfaces of the column. Spaces about $\frac{1}{16}$ inch wide were left round the pins, so that a small change could take place in the length of the main bar without the movement of the pins being interfered with by their coming against the concrete.

In taking the measurements, a Martens mirror-apparatus was attached to one of the pairs of pins, and a second one on the surface of the concrete parallel to and near the line between the pins. In this way it seemed possible to measure the longitudinal strain of the concrete, and independently of this, the strain of the same length of steel bar. The gauge-pins were made rigid enough to render it possible to find out the strain of the bar between the pins by taking measurements between their ends.

The readings appeared sufficiently good to warrant the employment of this method in the larger experiments, as the increments of strain, by their similarity for the concrete and the steel, indicated that the two materials were strained approximately to the same extent for the same loads.



Figs. 1. Details of Experimental Column.

Design of the Experimental Columns.

The Author's intention was to carry out experiments on columns similar in design and material to those used in the best practice. The precise details of the design are due to the kind co-operation of the late Mr. Hubert Congreve, M. Inst. C.E., Chief Engineer to the Manchester Ship-Canal Company. It was Mr. Congreve who first suggested to the Author the desirability of obtaining the information which these experiments sought to provide; and, after discussing the latter's suggestions as to a method for measuring the strains, he kindly undertook to provide a design for the experimental columns based on his usual practice and embodying the device for the measurement of the strains.

The design of the experimental columns is shown in *Figs. 1.* The main portion of each column consisted of a concrete prism, 6 inches square in section, and reinforced longitudinally by four round steel bars $\frac{3}{4}$ inch in diameter. These bars were tied by transverse stirrups, consisting of twisted loops of $\frac{3}{16}$-inch steel, placed 5 inches apart. The parallel portion of the column constructed in this way had a length of 6 feet, and it was completed by having enlarged ends 9 inches square and 1 foot in length, giving a total length of 8 feet. The longitudinal bars were allowed to penetrate to within 2 inches of the ends, which had an additional reinforcement in the shape of coils of $\frac{3}{16}$-inch steel wire. The ratio of length to diameter was 16 to 1, taking the columns over all, and 12 to 1 for the parallel part alone, so that they were short enough to be taken as short columns and for the bending effect to be neglected; that is to say, they could be treated for simple compression alone.

The gauge-pins are indicated in *Figs. 1*, and on the cross section are marked the reference numbers of the several positions of the Martens measuring-bars.

Composition and Construction of Columns.

The materials used in the construction of the concrete were:—

Broken Penmaenmawr granite, screened to pass a $\frac{3}{4}$-inch mesh and to be retained by a $\frac{1}{2}$-inch mesh . . .	27 cubic feet.
Leighton Buzzard sand, unscreened	$13\frac{1}{2}$ „ „
Portland cement	$6\frac{1}{2}$ cwt.

This gives a concrete containing about 5 parts of stone and sand

ballast to 1 part by volume of cement. Tests on the cement used yielded the following results:—

Fineness, residue on 180-mesh sieve	17½ per cent.
„ „ 80 „	2·9 „
Tensile strength, neat, after 7 days	560 lbs. per sq. in.
„ „ „ „ 28 „	640 „ „
Time of setting hard	1 hour 50 minutes.

From each mixing of the concrete used in the construction of the columns two 9-inch cubes were made for testing, and the following results were obtained from these:—

Cubes from Column.	Age.	Compressive Strength.			
		First Crack.		Crushing.	
	Months.	Lbs. per Sq. In.	Tons per Sq. Ft.	Lbs. per Sq. In.	Tons per Sq. Ft.
B	3	2,180	140	2,780	179
C	3	1,570	101	1,910	123
	3	1,520	98	1,870	120
D	6	2,560	165	2,640	170
	6	2,980	192	2,980	192
E	6	2,180	140	2,260	145
	6	2,140	138	2,360	152
F	6	1,520	98	1,880	121
	6	2,115	136	2,180	140

In the case of column A the cubes were omitted.

The observations necessary for the finding of the elastic modulus of the concrete were made upon a prism, 6 inches square and 32 inches long, having the same composition as the columns. The experiments were made in a horizontal 50-ton testing-machine with a Martens mirror-apparatus for taking the elastic measurements. In carrying out the experiments the prism was loaded up to about 400 lbs. per square inch, and three sets of readings were taken for each of the following results:—

Young's modulus of elasticity, at 2 months	1,220,000 lbs. per sq. in.
„ „ „ 5 „	1,535,000 „ „

The longitudinal reinforcement consisted of ¾-inch round mild-steel bars. These were not treated in any way, and were used just as they left the mill, with a slight rusting caused by their being exposed to the damp atmosphere.

The following results were obtained from tests of the steel:—

Limit of proportionality	12·5 tons per sq. in.
Yield-point	18 „ „
Maximum stress	26 „ „
Elongation on 10 inches	30·2 per cent.
„ 8 „	32·1 „
„ 2 „	47·5 „
Reduction in area	65 „

The modulus of elasticity of the steel, as determined from readings taken with a Martens and also with a Ewing extensometer, was found to have an average value of 30,200,000 lbs. per square inch.

The columns were built in wooden moulds or forms, which were held in a vertical position during the operation of filling and while setting. When the steelwork had been fixed in position and the mould was closed, filling commenced. In order to facilitate the work, the mould was constructed so as to be completely closed on three sides by continuous boards, while the fourth was made up of a number of separate pieces of timber, each 12 inches deep, which were added as the concrete rose. The whole column was thus certain to be more uniform and sound than if the filling had been done in one operation from the top.

The moulds were left in the vertical position and allowed to remain in a somewhat damp atmosphere for 14 days; after which, the boarding having been removed, the columns were kept in a damp position until ready for testing. During this last period they were frequently sprinkled with water.

When the moulds were ready for filling, with the steel in place, short lengths of thin iron tubing were slipped on to the gauge-pins, fitting suitable holes in the wooden sides of the moulds. On removing the woodwork these tubes were withdrawn, leaving annular spaces about $\frac{1}{8}$ inch wide round the pins. In the case of two of the 8-foot columns the gauge-pins were forged on to the main bars, and in the other two they were fitted into holes drilled for the purpose.

The Experiments.

The main experiments were carried out in a horizontal Buckton testing-machine, the ends of a column being bedded in plaster of Paris with thick layers of cardboard between it and the platens. Each column was set in the machine with two of its faces vertical; and one of these faces, containing the ends of the gauge-pins, was placed so as to face towards the observer, who took the readings through the telescopes.

As only two Martens mirrors and two telescopes were available, it was necessary to take four sets of readings up to the working-load, one for each corner of the column. In each of these sets one Martens measuring-bar was applied to a pair of gauge-pins and the other to the concrete midway between the two pairs of pins and on the same face.

For purposes of reference the positions of the measuring-bars on the pins and concrete were numbered as indicated in *Figs. 1.* It will be seen that the pairs of pins are marked respectively 1, 2, 3, and 4; while the positions of the measuring-bars on the concrete are 5 and 6, at the front and back of the column respectively. In the four sets of readings up to the working-load observations were taken on (1) and (5), (3) and (6), (4) and (5), and (2) and (6), in that order. Undoubtedly the ideal plan would have been to place mirrors in all six positions, and to take simultaneous readings at the specified loads; but as two mirrors only were available, the observations had to be taken in successive pairs as described. It was thought that, as there is no permanent set in the steel below the working-load, and very little in the concrete, readings taken successively in the manner described would differ little from similar observations taken at the same time.

The columns were designed to carry a working-load of $13\frac{1}{2}$ tons, and it was up to this load that the first set of readings were taken, with the mirrors at (1) and (5). The load was varied by increments of $\frac{1}{2}$ ton, and mirror readings were taken after each new setting of the load. On reaching $13\frac{1}{2}$ tons the load was removed and any permanent set was noted. As it was necessary to keep at all times sufficient load on the columns to prevent slipping and consequent disturbance of the mirrors, the zero was taken at 1 ton, and the load was brought back to this point after the completion of each set of readings.

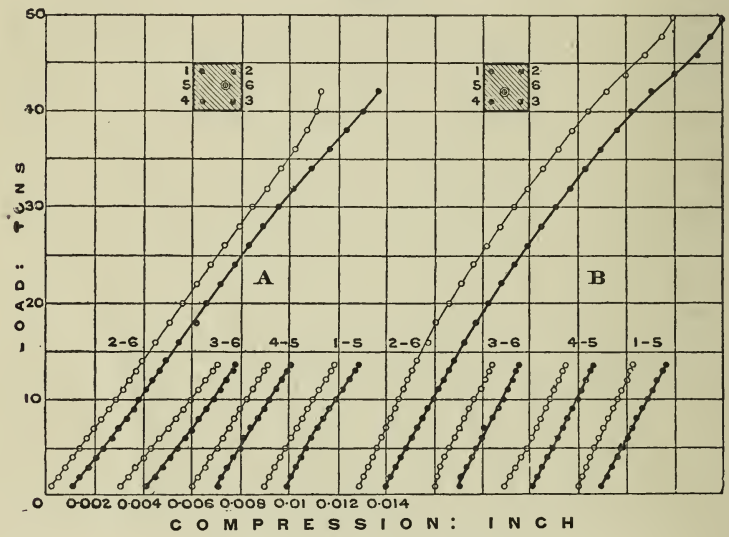
When the first set had been taken the mirrors were removed and readjusted in their second position, generally the one diagonally opposite the first, and the second set of readings was proceeded with.

In the case of the fourth set the load was taken beyond the $13\frac{1}{2}$ tons, by increments of 1 ton, until either failure occurred or the limit of the machine was reached. As the main object of these experiments was not destruction, but the acquisition of some definite knowledge of the behaviour of the steel and the concrete under working conditions, the columns were made as large as possible, so as to be comparable with columns in practice. The result of this policy was that one only of the four 8-foot columns

collapsed, the remaining three withstanding the 50-ton load of the machine.

The results of the experiments are given in the curves of *Figs. 2, 3* and *4*. These comprise the graphic records of the experiments on the four 8-foot columns, A, B, D, and E. Columns C and F, referred to later, were shorter. In *Figs. 2* and *3* are shown four pairs of curves for each column; those which have their points indicated by open circles refer to the steel and those with blackened circles to the concrete. At the commencement of each curve is placed the number corresponding with the position on the column

Fig. 2.



Readings for Columns A and B.

where these particular readings were taken, as shown on the cross sections drawn on these figures.

Column A (3 months old).—Readings were first taken on (1) and (5); and it will be seen that the plotted lines for these positions are slightly curved, but the same distance apart throughout, which suggests that under its first loading the concrete did not quite follow the straight-line law. It showed a growing increment of compression, with the result that more stress was thrown on the steel, and this was followed by a corresponding curvature of the steel line.

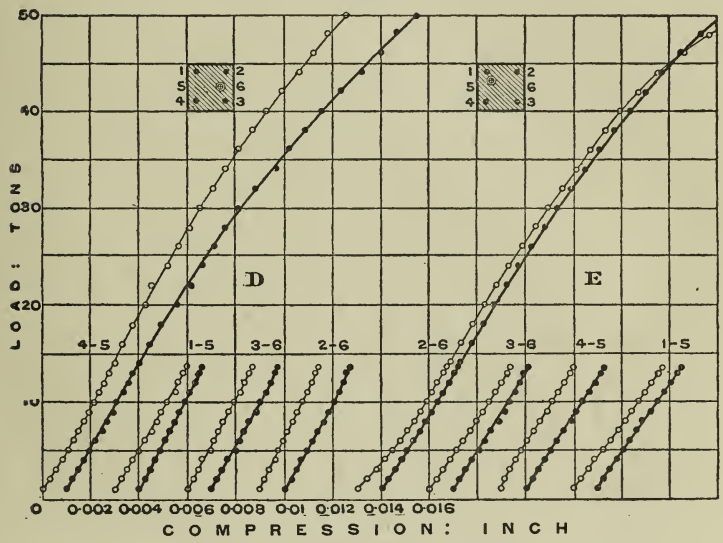
The second pair of points, (3) and (6), are nearly parallel, but have a flatter slope than the first two.

Next follow (4) and (5), which have the same slope as (1) and (5), but are straight instead of being curved.

The curves for the last position, (2) and (6), are similar to (3) and (6), and have about the same inclination.

The differences in slope of the various lines are, the Author thinks, partly due to eccentricity of the load-line. From the fact that (6) gives a flatter line than (5), it seems probable that in the present case the load line runs parallel to the geometric axis of the column, but somewhat nearer the back than the front, as indicated by the crossed circle in the cross section.

Fig. 3.



Readings for Columns D and E.

On the plotted diagrams for A, B, D, and E a sectional sketch of the column is shown, in which is indicated, by means of a double crossed circle, the probable position of the load-line. It must be clearly understood that the precise position of this line is not intended to be given but merely an indication of its possible whereabouts.

In the last of the pairs of curves the parallelism is maintained far beyond the working-load. The column actually collapsed at 44·5 tons.

Column B (*3 months old*).—The results with this column are not unlike those obtained for column A; the slight divergence of two

of the pairs of lines suggests that the load-line is a little below the centre, but half-way between back and front, as shown. Lines (1) and (2), which were nearer to the top of the column, are steeper than the others, showing that they were carrying smaller stresses for given loads. In the fourth set the divergence ceases after the 16-ton load has been passed. This seems to point to greater yielding on the more heavily loaded side having had the effect of restoring a balance in the stress. This column was loaded to 50 tons without failure.

Column D (*6 months old*).—The curves of this column are similar to those of A and B. Taking the steel and concrete together there is just enough difference between the slopes of front and back lines to suggest the position indicated for the load-line, though it might have been rather nearer the centre.

In the last of the four sets, (4) and (5), the lines for the steel and concrete run almost quite parallel to points well beyond the working-load, after which there is a slight divergence. It will be noticed that the concrete line curves somewhat, becoming steeper as the working-load is approached, and this is responded to by the steel.

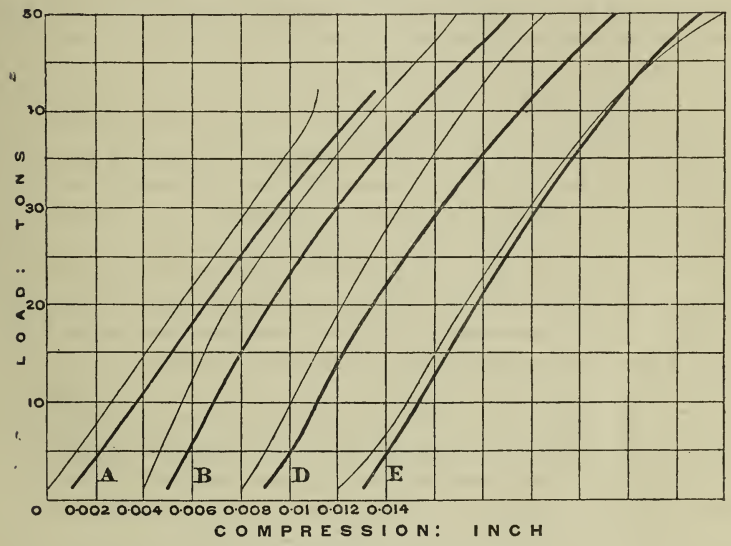
Column E (*6 months old*).—The curves for this column appear to differ from all the others. The concrete lines show (5) to be flatter than (6), while of the steel lines (1) and (2) are flatter than the others. In (2) and (6) the increments at the commencement are greater for the steel than the concrete, gradually becoming less until, at 6 tons, they are equal, and remain so up to about 25 tons. The reason for this is not clear. For some reason the steel at the beginning is carrying more than its share of stress. It might be suggested that the effect of the three first loadings and releases has been to disturb the grip, which was not again fully taken up until the load of 6 tons had been reached.

Comparison of Results.—For the purposes of comparison the pair of curves for the last set of readings on each column are shown replotted in *Fig. 4*. Here it will be seen that the general slopes of the curves for B, D, and E are practically the same, while that for A is a good deal flatter. This might be due to a difference in the mixing of the concrete causing a lower elastic modulus, which in turn would give rise to a lower relative stress in the concrete for a given strain, followed by an increased stress in the steel, and this again would be followed by a flatter line for the steel. In this particular column the broken stone was not screened so as to go between the $\frac{3}{4}$-inch and the $\frac{1}{2}$-inch meshes, but all that would pass the $\frac{3}{4}$-inch screen was put into the concrete, including the stone dust. In the other three this small, broken stone and dust was replaced by sand.

It must be noted that this column collapsed before the 50 tons had been reached, which also suggests a difference in the material: on the other hand, it is seen that the parallelism of the curves is the most nearly perfect of all.

In order to be able to see how nearly the mean readings gave similar increments of strain, they were calculated for all the loads of each column, and the resulting mean values are plotted in *Fig. 5.* Columns A and D gave quite parallel lines for the steel and concrete. In column B there is a slight divergence; and in

Fig. 4.



Last Readings Replotted.

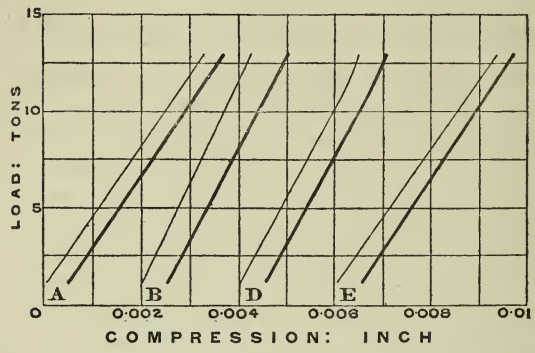
column E the lines converge somewhat. The last result is no doubt due to the abnormal curves in the fourth set of readings.

In interpreting these curves it must be remembered that any movement of the concrete past the steel would result in a flat concrete line, and a steeper steel line. But both this and the converse happened in the case of a single column. Sometimes it occurred in one column, sometimes in another. Moreover it must be remembered that the strains represented on the diagrams are of very small magnitude. At the working-loads of the columns the total strain, for both the steel and the concrete, amounts to only about $\frac{3}{1,000}$ inch on a length of 12 inches, or $\frac{1}{4,000}$ of the

measured length. If any slipping were taking place a larger movement than this might be expected.

Another point which might help to account for the small convergences and divergences of the plotted lines is to be found in the fact that the measurements on the concrete were not taken quite close to the steel, but some little way from it. This was unavoidable,

Fig. 5.



Mean Values.

and it might well be that the concrete whose shortening was being measured was carrying more, or less, stress than that part which closely surrounded the bar in question.

From a consideration of these several points the Author thinks it reasonable to form the opinion that in these experiments no appreciable movement of the concrete on the steel took place.

Distributed and Concentrated Loads.

Column F was utilized for experiments in which the load was at one time uniformly distributed over the ends in the usual way, and at another time was transmitted through what were virtually points in the centres of the ends. This particular column had a total length of 5 feet.

Four sets of readings only were taken, namely, concentrated, (1) and (5), then (3) and (6); distributed, (1) and (5), then (3) and (6).

In the case of the concentrated loading it might have been expected that, with no direct pressure transmitted to the ends of the bars, they would be partly forced through the concrete and fail to carry their share of the load. The results of the experiments when plotted, however, showed that this was by no means the case,

and that the concrete held on to the steel so as to carry its share of the load as if this had been directly transmitted to it.

The second of the 5-foot columns (C) was tested to destruction and was found to fail at 50·75 tons. The intention in making these two last columns shorter than the others was to enable them to be placed in a large compression machine and so to ensure their destruction if necessary.

Experiments on the Frictional Resistance of Bars.

When the concrete was being mixed for the purpose of building the columns, the Author took the opportunity of making six reinforced-concrete prisms for the purpose of determining the loads causing the steel to slip in the concrete. These prisms were 4 inches square and 42 inches long, and in each was embedded for a known distance a length of the $\frac{3}{4}$-inch steel used in the columns. The depths of insertion ranged from 40 to 10 inches, decreasing by uniform differences of 6 inches. A sufficient length of steel was left projecting from the prism to be taken hold of by the tension grips of a testing-machine, and the end of the prism was allowed to press against the back of the other crosshead of the machine, the bar at the same time being allowed to pass through a hole in this crosshead. The pull was applied by the machine and increased until such time as some sign of failure was visible. During the application of this load the yield-point of the steel was sought for by means of a pair of dividers.

In the case of the more-deeply embedded specimens the first sign of failure to be noticed was the yield-point of the steel, that part of the bar within the concrete remaining firmly embedded meanwhile and showing no sign of movement. The effect of the yielding of the steel outside the concrete was to cause a diminution of its diameter, with the result that the concrete began to lose its mechanical grip, this failure beginning at the point where the steel entered the concrete, and gradually spreading inwards. When this thinning, and consequent relaxation of the grip, had penetrated a certain distance, the length of the steel held by the concrete getting shorter and shorter, the bar was found to pull out suddenly. At the point where this happened the tension on the bar was just sufficient to overcome the frictional grip of the concrete, and it was found that the part of the steel which had been sliding through the concrete still retained its original diameter. The load required to pull out the steel in this way was limited by its yield-point, and was found to vary very little. After the experiments the bars were

carefully measured, and it was found that for a certain distance from the end which was embedded in the concrete the bar was parallel and of its original diameter. From the end of this part to the grips of the machine there was a uniform reduction corresponding with that produced at the yield-point of the material.

This appears to show very clearly that the grip of concrete on steel in tension cannot be maintained for long after the steel has begun to yield. With compression bars the stress on the steel is always increasing its diameter and tending to tighten the grip. The results of this experiment are given in the following Table:—

Frictional Resistance to Slipping.

Specimen.	Age.	Depth of Insertion.	Yield-Point of Steel.	Load at which Slipping Occurred.	Length of Slipping Portion.	Length of Slipping Portion in Diameters.	Gripping Force per Unit Area of Bar.
	Months.	Inches.	Tons.	Tons.	Inches.		Lbs. per Square In.
1	2½	40	7·50	7·50	18·0	24·0	396
2	2	34	7·75	8·95	15·8	21·0	538
3	2½	28	7·85	8·75	13·5	18·0	614
4	7	22	7·96	7·96	22·0	29·0	343
5	7	16	..	7·20	16·0	26·4	427
6	7	10	..	2·57	10·0	..	243

Conclusions.

The working-load on the columns has been taken at 13½ tons; the combined area of the cross section of the four steel bars, $a = 1{\cdot}77$ square inch; the area of cross section of the concrete, $A = (36 - 1{\cdot}77) = 34{\cdot}23$ square inches; and the ratio of the elastic modulus for the steel to that of the concrete was—

$$m = \frac{30{,}200{,}000}{1{,}535{,}000} = 19{\cdot}8.$$

Inserting these values in equations (1) and (2), given at the beginning of the Paper, the stress in the concrete, C, is found to be 437 lbs. per square inch, and that in the steel, S, is 8,656 lbs., or 3·86 tons per square inch. That is to say, each of the bars was carrying about 1¾ ton, which load, when compared with the figures in the last Table, was obviously nothing like sufficient to cause a movement of the concrete in the steel.

In proceeding to draw any general conclusions from the results of these experiments, it is to be borne in mind that they were carried out primarily for the purpose of ascertaining directly the distribution of stress in reinforced columns, and at the same time of learning to what extent, if any, slipping took place between the two materials. The scope of the experiments was limited and they were comparatively few in number. The Author considers, however, that the following conclusions may reasonably be drawn:—

The method used for the measurements of strain in the steel and the concrete was satisfactory, and, so far as could be seen, the cutting of the holes in the concrete to allow of the penetration of the pins to the surface did not appear to have influenced the results.

Under the conditions existing in the columns experimented upon there is no evidence of movement of the concrete relatively to the steel.

The fact that the steel bars did not come through the pillar so as to take their load directly from the platens of the machine made no difference, the grip of the concrete on the steel being such that the method of calculating the relative stresses, on the assumption that the ends of the bars are flush with the surface of the concrete, applies equally well to cases like the present where the bars are held in the grip of the concrete.

It is highly probable that the small differences of slope in the plottings for the concrete and steel are due partly to slight eccentricities of loading and partly to local differences in the quality of the concrete.

The conclusion that there was no slipping taking place is supported independently by the experiments on frictional grip.

It is interesting and important to notice how in a column constructed on this system the load on the steel adapts itself according to the elastic properties of the concrete employed. With a concrete of low elastic modulus (such as the column A) the concrete suffers relatively greater compression than if it had a high modulus, and by doing so throws more load on the steel; the result of this is relatively greater compression of the steel, and a flatter line in the plotting. The effect is to produce two parallel lines for the steel and concrete, but flatter than they would have been with a concrete of higher modulus.

Finally, the results of these experiments appear to show that in concrete columns reinforced by longitudinal plain steel bars, and constructed with reasonable care, there is considerable margin between the loads carried by the steel bars and the loads which

would be required to force them through the concrete. It is therefore fair to make the usual assumptions which have been referred to in discussing the method of calculating the stresses in the steel and in the concrete.

The foregoing experiments were carried out in the materials-testing laboratory of the Manchester School of Technology, and the Author wishes to thank the City Education Committee, Principal Reynolds, M.Sc., and Professor J. T. Nicolson, D.Sc., M. Inst. C.E., for the facilities afforded in carrying out the experiments. In conclusion, he desires to express his sense of very deep regret and of personal loss caused by the death of Mr. Hubert Congreve, M. Inst. C.E., with whom it was his privilege to be associated both in this and in other experimental work.

The Paper is accompanied by ten tracings from which the Figures in the text have been prepared; also by three photographs.

(*Paper No. 3966.*)

"Composite Columns of Concrete and Steel."

By William Hubert Burr, M. Inst. C.E.

The effect of a concrete filling on increasing the carrying-capacity of a steel column has never been sufficiently investigated; accordingly the tests here described were made by the Author in order to ascertain the relative strength of plain steel columns compared with composite columns consisting of an outer frame of steel compression members and bracing filled with concrete.

These columns were tested at the testing-laboratory of the civil engineering department of Columbia University, New York, in June, 1908. It was not the intention to secure such refinement of results as might be obtained in a physical laboratory with specimens of small dimensions and of simple character, but rather to make both the plain-steel and the reinforced-concrete columns as nearly as possible similar to those which the engineer uses in practice.

The Two Types of Column.

In investigating the carrying-capacity and other physical characteristics of reinforced-concrete columns, the conditions involved in actual construction should be carefully kept in view. There are at present what may properly be considered as two classes of reinforced-concrete column.

The first class, that generally employed, is designed with the steel used both as a wrapping or banding material, and in comparatively small rods placed parallel to the column axis near to the bands. In some cases these small longitudinal rods may be secured to the wrapping material by steel wire, or they may even be used as a kind of cage, with a comparatively thin layer of concrete or mortar separating them from the enwrapping steel. In columns of this type the reinforcement is not so distributed or held as

to constitute a direct load-carrying element of the column; its main purpose is to support the concrete by enwrapping it so as to prevent as far as possible failure due to compression, usually attended by shearing, as it would occur in plain concrete. The longitudinal steel rods may be capable of resisting by themselves some compression, but it is of such small amount as to exert little or no direct material influence upon the carrying-capacity of the member. Indeed the rods usually employed are so easily bulged or crippled laterally by small compressive loads that they would certainly be sources of weakness in the reinforced column if it were not for the spiral or other enwrapping steel. It has been shown by Professor A. N. Talbot and others that with this type of reinforcement the concrete receives little support from the reinforcing steel until the load is nearly equal to the ultimate resistance of plain concrete. In other words, while such reinforcement adds to the carrying-capacity of the concrete, it does not make its influence felt substantially with the progressive application of increasing loads until such limit has been nearly reached. It is obviously not well adapted for effective use in long columns, nor for high buildings where continuous columns frequently reach up through fifteen, thirty, or even forty stories.

The other class of column is distinguished by a type of steel reinforcement which is a direct load-carrying member in itself, while at the same time it effectively enwraps or bands the concrete so as to give it the greatest possible added carrying-capacity which any kind of banding can produce. In such columns the steel reinforcement is self-supporting, and it may be as light as can be made with the smallest shapes rolled, or may be as heavy as the demands of any special piece of work require. Such columns are adapted to buildings of any height whatever, and to any length of long-column design which may be required. The concrete is so closely banded by the steel column, and both parts of the member are so intimately and rigidly bonded, that each column must necessarily act as a firm unit, so that the steel reinforcement gives effective support to the concrete with every progressive addition of loading, from the lowest to the highest. In such columns the reinforcing steel member must have external dimensions of cross section nearly as large as those of the finished column, the outer thickness of mortar or other encasing material being just sufficient to give the requisite protection against fire or other sources of damage. This outer concrete or mortar is simply a protecting shell, only the concrete lying within the steel being considered as load-carrying. This is necessary because in case of fire

much of the outside or protecting material may be destroyed or knocked off, so that the carrying member should be considered to consist only of the steel reinforcing column and the concrete lying within it.

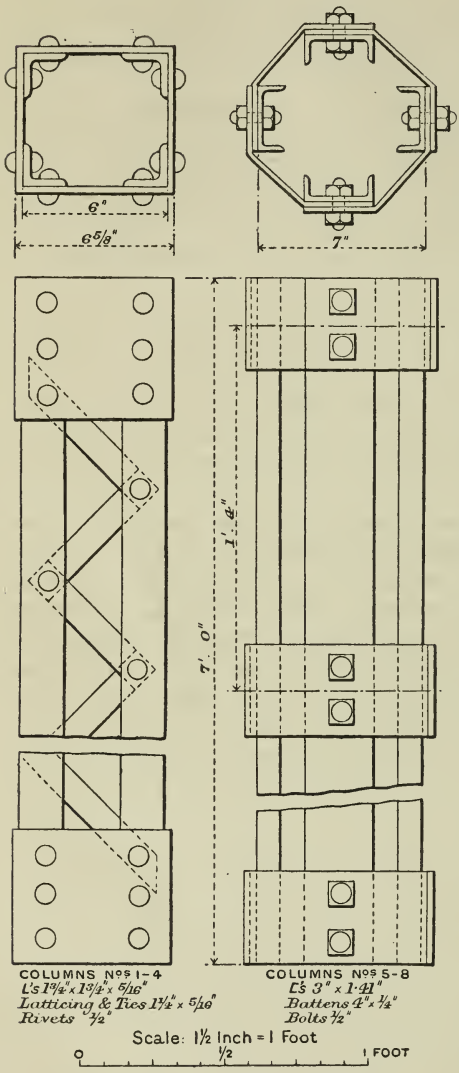
This latter type of reinforced-concrete column was used in the design by the Author of what is known as the 39th Street Building, in New York City, a building occupied by the McGraw Publishing Company and other publishers. It has been used partially above the second floor for the operation of large printing-presses. The same type of reinforcement on a large scale will be used, also under the design of the Author, for the great 725-foot span, reinforced-concrete arch of the Henry Hudson Memorial bridge, which it is proposed to build across Spuyten Duyvil Creek in the northern part of the city of New York.

Description of the Columns Tested.

Four of the second class of reinforced-concrete columns were made for testing, and for comparison four similar steel columns without the concrete were also tested. The steelwork of these eight columns followed two designs (*a* and *b*, *Figs. 1*), there being four columns of each design, of which two were tested with and two without concrete. The 3-inch rolled-steel channels in design (*b*) weighed 4 lbs. per lineal foot. The total cross section of the four steel angles was 4 square inches, while the aggregate sectional area of the four 3-inch channels was 4·76 square inches. The steel angles were placed at the corners of a square, each side of which measured 6 inches, and were held in place by $1\frac{1}{4}$ by $\frac{5}{16}$-inch lattice bars with $\frac{1}{4}$-inch batten-plates at each end. The width between the exterior lattice bars filled with concrete was $6\frac{5}{8}$ inches, but the precise dimensions of cross section for purposes of computation cannot be stated exactly. Hence the cross section of the reinforced column as a load-carrying member will be taken as $6\frac{1}{2}$ inches square. The opposite channels of the other four columns were so placed as to be 7 inches from back to back, and were connected with 4-inch by $\frac{1}{4}$-inch batten-plates, so bent as to enwrap the four channels and make the finished column of octagonal form. The exterior dimensions of the steel in these four columns is $7\frac{1}{2}$ inches. The enwrapping batten-plates were 16 inches apart from centre to centre along the length of the column.

The four reinforced-concrete columns were built with the care given in the best constructional practice, as to both the steel and the concrete, but no more. Their total length was 7 feet.

It is evident that these columns are much smaller than those ordinarily used in reinforced-concrete construction, although they may be considered as coming within the minimum sizes of such structural columns. The circumstances of their manufacture and their dimensions, it is believed, give the results of the tests a significance applicable to actual reinforced-concrete columns of this type used in building or other construction.



STEELWORK OF COLUMNS TESTED.

The concrete used in all the columns was composed of 1 volume of Alpha Portland cement, 2 of excellent coarse sand, and 4 of trap rock, crushed small enough for the largest pieces to pass through a 1-inch ring, and by far the greater part of it to pass through a $\frac{3}{4}$-inch ring.

The ends of the steel reinforcing columns were carefully milled, to the exact length, at right angles to the axis of each column. The ends of the reinforced-concrete columns were flushed with neat Portland cement immediately before being placed in the testing-machine, so as to make an even bearing as nearly as possible over every part of each end.

Results of Tests.

The tests were made in an Olsen testing-machine of 400,000 lbs. capacity, the heads of which were fitted with hemispherical bearings so as to ensure accurate centre loading, and as nearly as possible perfect distribution of the applied load.

All compressions were measured by a compressometer reading to 1/10,000 inch, and the instrument was so set as to read compressions on an accurately measured distance of 70 inches. The lateral motions due to flexure were measured by screw micrometers, reading from rods held in place by the upper and lower frames of the compressometer; and care was taken to secure measurements in normal directions.

Curves showing compressions and lateral deflections were constructed from a complete set of observations, and are given for all the columns in *Figs. 2–5*. They show the deformations and other physical features of the tests as they progressed from the initial loading up to failure. It may be mentioned that the various set curves represent permanent set of the columns after removal of loads.

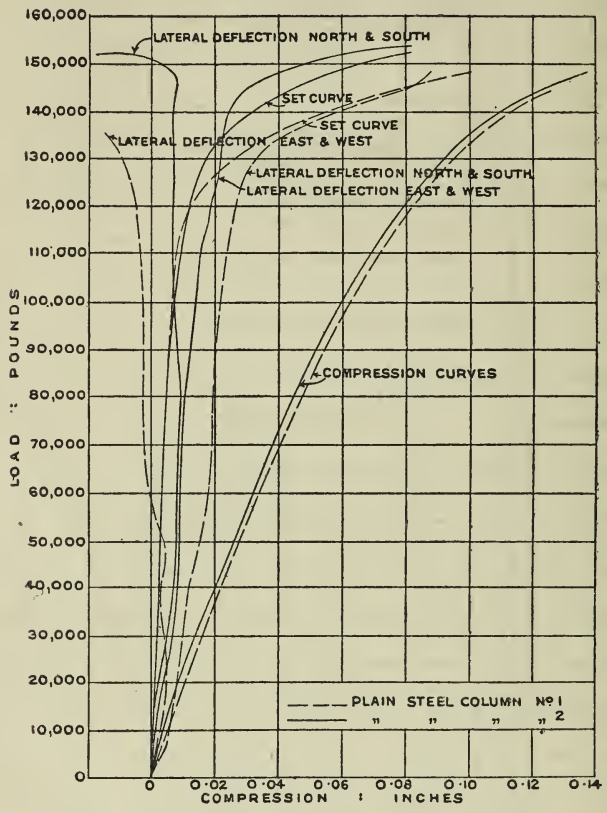
In order that full information regarding all the elements of resistance of these columns might be secured, the four plain steel columns, duplicates of the reinforcing steel in the other columns, were tested simply as steel columns. These columns are numbered 1, 2, 5, and 6, as indicated in the diagrams. The length divided by the radius of gyration was 34 for the angle-bar columns and 31 for the channel-bar columns; it may at once be observed that they were short steel columns and not long members. Curves showing the results of tests on these columns are given in *Figs. 2* and *4*, and the following Table shows the ultimate resistance obtained:—

Plain Steel Columns.

Column.	Type.	Sectional Area.	Ultimate Load.		Length.	Ratio l/r.
No.		Sq. Ins.	Lbs. per Sq. In.	Tons per Sq. In.	Inches.	
1	Angle (*Fig. 1, a*)	4·00	37,050	16·5	84	34
2	„	4·00	38,000	17·0	84	34
5	Channel (*Fig. 1, b*)	4·76	32,600	14·5	84	31
6	„	4·76	32,140	14·3	84	31

From this it will be seen that Nos. 1 and 2—the angle-bar columns with the usual lattice bracing—show ultimate resistances at failure distinctly higher than Nos. 5 and 6—the channel-bar

Fig. 2.

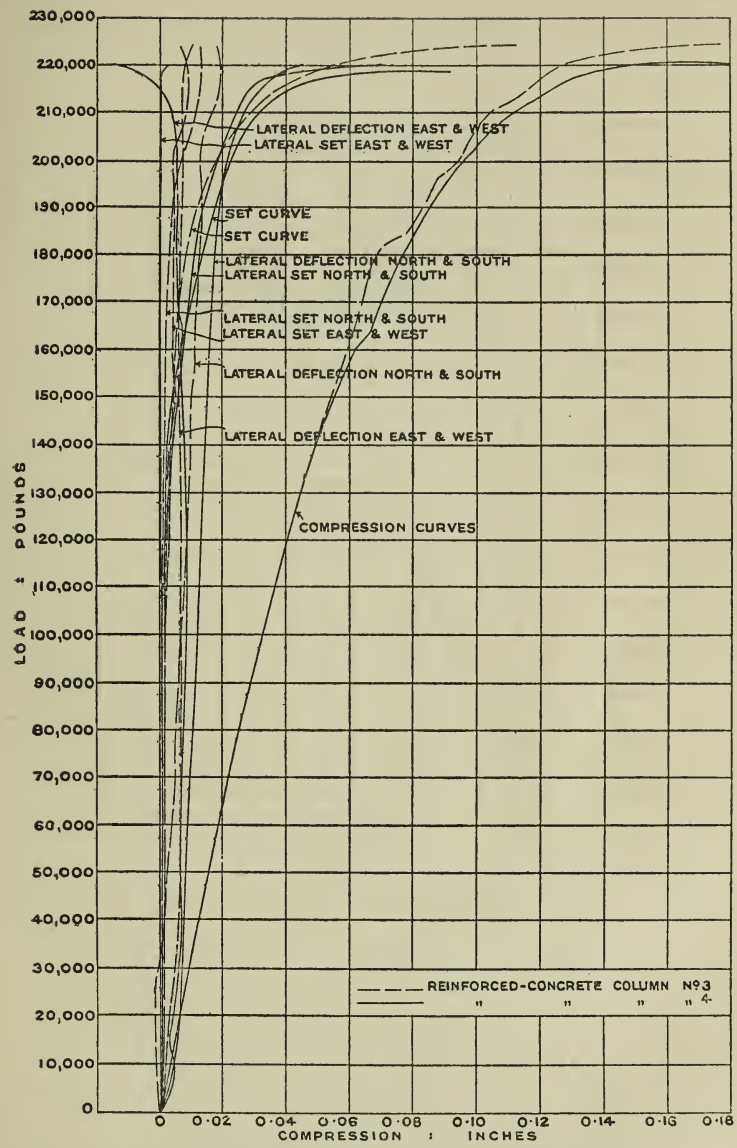


Tests of Two Plain Steel Columns (*Design a, Figs. 1*).

columns without positive bracing between them, but with battens only. The advantage of holding the members of a column under compression by complete effective bracing is thus apparent.

Specimens of the steel taken from the angles and channels of these columns showed an average yield-point of about 40,800 lbs. per square inch, and an ultimate tensile strength of about 61,300 lbs. per square inch. No effort was made to secure any

Fig. 3.

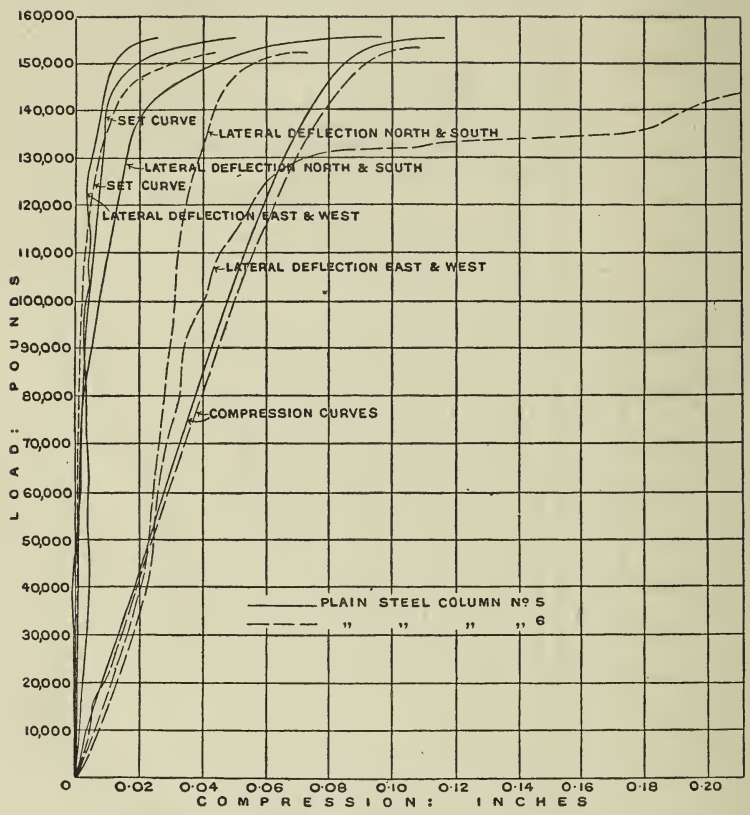


TESTS OF TWO REINFORCED-CONCRETE COLUMNS (*Steelwork of Design a, Figs. 1*).

special quality of structural steel, but the results show that it was a good medium material.

The concrete in the reinforced columns was 3 months old when the columns were tested. It was mixed by hand, reasonable care being taken to secure a thorough mixture, but nothing more, and

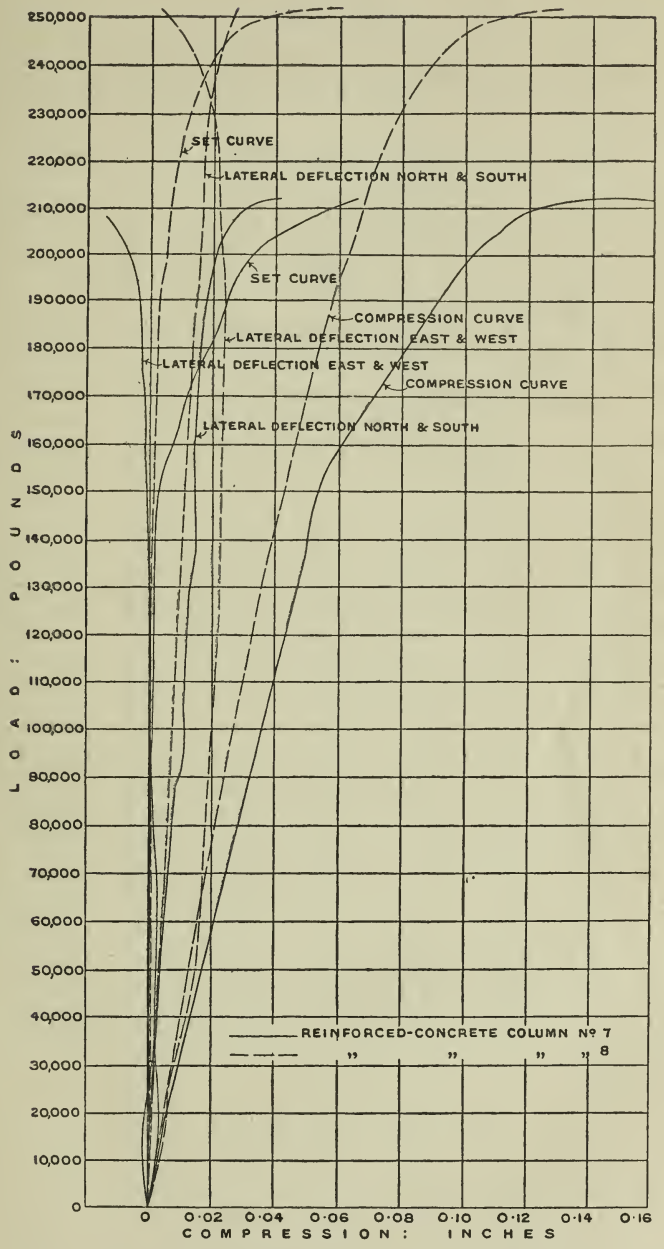
Fig. 4.



Tests of Two Plain Steel Columns (*Design b, Figs. 1*).

the concrete was therefore a fair material corresponding closely with that used in ordinary first-class work. The columns were kept in a vertical position from the time when the moulds were filled to the time of testing. They were frequently wetted, especially in the earlier part of the 3 months.

Fig. 5.



TESTS OF TWO REINFORCED-CONCRETE COLUMNS (*Design b, Figs. 1*).

Calculations.

In discussing analytically the results of the tests of the four reinforced-concrete columns the following notation will be employed:—

E_s denotes the coefficient of elasticity for steel = 29,000,000 lbs. per square inch.
E_c „ „ coefficient of elasticity for concrete (to be found).
A_s „ „ area of steel section in square inches.
A_c „ „ area of concrete section in square inches.
A „ „ $A_s + A_c$.
Δl „ „ shortening of column or compression for 70 inches length as found in tests.
E „ „ coefficient of elasticity for combined section so that $A E = A_s E_s + A_c E_c$.
P „ „ total load on a reinforced-concrete column, as found by test.

These total loads, P, are obviously taken from the test records shown on the curves.

The coefficient of elasticity for steel is taken at 29,000,000 lbs. per square inch as that is a fair and close value of the quantity for the mild structural steel used in the columns. The coefficient of elasticity for concrete, as is well recognized, is not nearly so well defined, since it depends upon the age of the concrete and the thoroughness of mixing, as well as upon the proportions of the cement and aggregate and upon other conditions attending the making of the concrete. Inasmuch as these reinforced-concrete columns were about 3 months old, the coefficient of elasticity for the concrete is somewhat less than that which it would attain at a greater age, although in most cases its further increase in value would not be great. It is to be remembered, in connection with such tests as these, that the coefficient of elasticity for concrete at an age not greater than about 3 months, and sometimes at much less age, must be depended upon for load-carrying conditions in the progress of construction of almost any reinforced-concrete structure.

In general the following equation may be used for the determination of E_c:—

$$A_s E_s \frac{\Delta l}{70} + A_c E_c \frac{\Delta l}{70} = P \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Since Δl is given by the test curves, all quantities are known in equation (1) except E_c, which can be evaluated at once for any particular loading for a given test.

Results of Tests.

On examination of the curves in *Fig. 3* it will be clear that the reinforced columns Nos. 3 and 4 reached what may be termed, approximately at least, their elastic limit at a load (P) of 180,000 lbs., Δl having the value 0·072 inch. Equation (1) however may be applied to total loads of 160,000 lbs. and 120,000 lbs. respectively, in order to discuss points certainly below the loading at which the columns show deficient elasticity.

Similarly values for columns Nos. 7 and 8 have been calculated from *Fig. 5* by taking total loads at 200,000 lbs., 160,000 lbs. and 120,000 lbs. It is seen, however, that column No. 8 shows a much better diagram than column No. 7, and an independent set of values has been used for that column at a loading of 200,000 lbs. Both these calculations have resulted in the figures given in the following Table. Average values of Δl for each pair of reinforced columns have been taken, except in the case of column No. 8 :—

Reinforced-Concrete Columns.

Total Load.	E_c	Stress in Steel.	Stress in Concrete.	E	Mean Crippling Stress on Total Section.
Lbs.	Lbs. per Sq. In.	Lbs. per Sq. In.	Lbs. per Sq. In.	Lbs. per Sq. In.	Lbs. per Sq. In.
		Columns 3 and 4.			
120,000	2,321,000	16,990	1,360	4,850,000	5,210 (Section = 42·25 sq. ins.)
160,000	1,850,000	24,850	1,585	4,420,000	
180,000	1,536,000	29,870	1,582	4,140,000	
		Columns 7 and 8.			
120,000	1,843,000	15,750	1,000	4,442,000	4,230 and 5,030 respectively (Section = 49·75 sq. ins.)
160,000	1,718,000	21,550	1,276	4,310,000	
200,000	731,000	33,930	855		
200,000[1]	1,648,000	26,100	1,483	4,270,000	

For columns Nos. 3 and 4, $r_c = 2{\cdot}26$; $\frac{l}{r_c} = 37{\cdot}2$; $\frac{l}{d} = 13$.

For columns Nos. 7 and 8, $r_c = 2{\cdot}4$; $\frac{l}{r_c} = 35$; $\frac{l}{d} = 11{\cdot}2$.

[1] Column No. 8.

The radii of gyration (r) have been computed for the combined concrete-and-steel normal column section. The ratio of column-length to radius of gyration has the values 37·2 and 35, only, showing that the columns, as such, are short rather than long. The ratios of length over exterior dimensions (d) of the column are also given for both pairs.

On the whole the results of the tests are not much different for the two forms of column-section, but what difference there is appears to favour the latticed-angle reinforcement. Somewhat higher values of both the coefficient of elasticity for the concrete and for the intensities of pressure are found for columns Nos. 3 and 4 than for Nos. 7 and 8. Similarly the coefficient of elasticity E for the combined material is rather higher on the whole for the same columns.

The modes of failure of the eight columns were the following:

Column No. 1, plain steel, failed by bulging of angles near one end.

Column No. 2, plain steel, failed partly by general flexure and partly by bulging of angles near one end.

Column No. 3, reinforced concrete, failed by the bulging of angles about 15 inches from one end; there was but little lateral deflection of the column as a whole. At the same time a fine vertical crack developed in the concrete near the top and at the centre of the column.

Column No. 4, reinforced concrete; this column showed initial failure at one corner at the end, although there was considerable general flexure. The angles bulged at one end and produced failure of the adjoining concrete.

Column No. 5, plain steel, failed by general flexure with deformation at centre.

Column No. 6, plain steel, also failed by general flexure.

Column No. 7, reinforced concrete, failed by bulging of channels near one end, causing a rupture of bond between steel and concrete.

Column No. 8, reinforced concrete, failed also by bulging of channels near the end destroying the bond between the steel and concrete.

These tests are too few in number, and the section of the concrete is too small relatively to that of the steel, to furnish a basis for any safe broad generalization, but it is believed that the results are significant as to certain general features of columns of this type. Among other things it would appear that it is advisable to design

the steel member by the same general principles which govern the design of the best class of steel columns, as in this manner the steel is given the best form both for effective banding and support of the concrete, and for the carrying of load by itself.

It also appears from these tests that it is abundantly safe and proper to take the coefficient of elasticity for the concrete at least as high as 2,000,000 lbs. per square inch, at the age of 3 months, and for such loads as are ordinarily imposed upon reinforced-concrete structures under proper specifications.

It would be interesting and of much value to continue the testing of similar columns for greater ratios of length to side, or radius of gyration, and at greater ages, in order to determine more completely, and therefore more satisfactorily, the ultimate compressive resistance of the concrete. It is believed that such tests would justify working-stresses as high as 500 to 750 lbs. per square inch in columns of this nature, and possibly higher stresses for structures of unusual magnitude.

The Author is indebted to Assistant-Professor J. H. Woolson and to Mr. J. S. Macgregor for devising details of testing apparatus and conducting the actual tests under his direction.

The Paper is accompanied by three diagrams, from which the Figures in the text have been prepared.

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Discussion.

The President. The PRESIDENT, in moving a vote of thanks to the Authors for the valuable information given in their Papers, remarked that Mr. Popplewell was present, but Mr. Ellis, who was on his way home from China, would not be able to attend during the discussion, nor would Mr. Burr, who was resident in the United States. He thought it was the first time that The Institution had had before it Papers dealing definitely with reinforced concrete. There was no doubt that there was very wide-spread interest at the present moment in work carried out in that material. In a certain sense it was a new material, involving new considerations—a material with regard to which the experience at present, although large in amount, was limited in many ways. There were a few engineers who had been engaged for some time in carrying out reinforced-concrete work, and who undoubtedly knew a great deal about it, but the members of The Institution did not wish to be entirely dependent on the few engineers who had had such special experience, and they desired to know, as far as possible, the results of other experience. Therefore The Institution heartily welcomed Papers which brought forward the results of experience and of experimental investigation. Reinforced-concrete construction was, of course, open to theoretical investigation, like other methods of construction, but he thought it was not doing it any discredit when he said that the calculation of the stresses in reinforced concrete involved rather more assumptions than did the calculation of the stresses in steel structures, with which engineers were more accustomed to work. The assumptions which had been made, and more or less tested, were no doubt good approximations, but it was very important that these assumptions should be re-tested from time to time. It was necessary to know the limits of approximation of the assumptions which were made in carrying out calculations relating to reinforced concrete, and experimental work such as that of Mr. Popplewell, who had tested some of the assumptions on which columns had been calculated hitherto, was very valuable. The Institution itself had foreseen the desirability of acquiring information about reinforced concrete for the use of its members, and it had appointed a Committee which was collecting information as to actual structures and as to the kind of assumptions that had been made in calculating the stresses in them. It had also carried

The President.

out for itself a certain number of experiments. Unfortunately, with a discrete body, like a reinforced-concrete column or beam, it was no use making experiments on a very small scale; they had to be made on a large practical scale, and consequently they were costly. Therefore The Institution was very glad to welcome Papers giving the results of experimental research. But good and valuable as experiments were, engineers did not like to depend upon them alone, and it was equally important therefore to obtain detailed descriptions of works that had been carried out, showing the devices adopted and putting forward some account of the results, good, bad, or indifferent, and of the accidents and the successes during the construction of such works. One of the Papers that evening was a very valuable one indeed in that respect. The subject altogether was one of great interest, and he hoped there would be a thorough discussion.

Mr. Popplewell.

Mr. W. C. Popplewell wished to emphasize the fact, which he had pointed out in his Paper, that the experimental work described was necessarily very limited in extent. The intention had been to find out something about the relative movement of the steel in the concrete—if there was any movement—and he thought the experiments had proved that, in the columns dealt with, at any rate, there was no perceptible movement. Another point was that the experiments on frictional grip had been taken in tension. Since carrying out the experiments he had made further experiments on frictional grip in compression, by pushing bars—ten $\frac{3}{4}$-inch bars and ten $\frac{9}{16}$-inch bars—through blocks of concrete of different thicknesses, and the results corresponded very nearly with those obtained in tension, the average frictional force exerted between the concrete and the steel being between 550 and 650 lbs. per square inch.

Mr. Meik.

Mr. C. S. Meik complimented Mr. Ellis upon the boldness of his design, having regard to the fact that he had placed the wharf upon piles the bearing-power of which he could not calculate by any recognized formula. It was certainly rather alarming to read (p. 7) that in one pier the piles sank rapidly under the steady weight of the hammer and cylinder, without any driving, and had to be checked by holding up this dead weight when they were at the required depth. He thought that any engineer who had to carry out works such as those described would have considered this not merely—as Mr. Ellis said—"unsatisfactory," but "extremely unsatisfactory." On the next page Mr. Ellis stated that the piles had stood the test of having more than the estimated load stacked upon them locally. Mr. Meik did not know to what extent this loading had been carried out, but it was apparent that on loading a

Mr. Meik. certain area of the wharf the piles had not shown any appreciable shrinkage. That demonstrated the advantage of reinforced-concrete decking. Such decking was of great assistance in distributing the load, and it therefore assisted the piles—especially such piles as those used in the present case—to carry a heavy weight. It would be very interesting to see how the piles behaved when the wharf came to be loaded over its entire area with the load it was supposed to carry. It was well known that a great deal of the carrying-capacity of a pile depended upon frictional resistance. Some little time ago he had to carry out a pier in Italy which was built on reinforced-concrete piles driven into sand, and he found that, after allowing a pile to rest for 2 months, the frictional resistance to driving had doubled. Farther on in his Paper Mr. Ellis referred to moulding the wharf-members on shore, carrying them into position, and joining them up. Mr. Meik had tried that plan lately for the first time, but he was bound to say it had not proved very satisfactory. There was great difficulty in getting the heavy concrete members into position and in satisfactorily joining them to the other work. In future he would certainly go back to the old system of moulding them in position, even if it were between high and low water. One of the chief advantages claimed for moulding on shore and carrying out was a saving of time, but, in his own experience, that advantage had proved illusory. Although Mr. Ellis gave the working-load of the steel as 16,000 lbs. per square inch, or about one-fourth of the ultimate stress, later on he mentioned the total load on a beam as being 93 tons, which, compared with its working-load of 14·6 tons, showed a ratio of $6\frac{1}{2} : 1$. He thought that what Mr. Ellis meant by the 16,000 lbs. was the maximum allowable working-stress, because the steel of many of the beams was not stressed to more than 12,000 lbs. per square inch. He thought the beams might have been designed rather more economically: in most of them there was 2 to 3 per cent. of steel, which was not a very economical ratio in dealing with work of this description. By making the beams a little deeper some of the steel might have been saved. He congratulated Mr. Ellis on his success in getting the work carried out at a very low cost. For instance, the reinforced-concrete foundations and ground floor of the warehouses had cost 2*s.* 3·4*d.* per cubic foot, which was very low. The cost of the piles in the wharf worked out at 3*s.* 7·4*d.* per cubic foot, which also was very low. In England it was not possible to get pile-work done under 5*s.* per cubic foot: perhaps the Chinese labour available, which was very adaptable in connection with such work, had resulted in some saving at

Shanghai; but it did not appear to account entirely for the very low cost of the work. Mr. Meik.

With regard to Mr. Popplewell's Paper, he heartily endorsed what the President had said about the advantage of the Paper to engineers. He noticed that there was considerable variation in the crushing-stress of the cubes of concrete. He presumed the cubes were made at the same time as the beams. There was a variation from 120 tons per square foot in the case of beam C to 192 tons per square foot in the case of beam D. Column C was said later on in the Paper to have been tested to destruction and found to fail at 50·75 tons, which was lower than most of the others. Variation in the strength of the concrete was one of the most difficult things that had to be contended with in designing reinforced-concrete structures. It was necessary, therefore, at the present time, to allow a considerable margin in connection with the ultimate strength. In the test-column referred to in the Paper the ratio of the volume of steel to the volume of concrete worked out at about 5 per cent., which for a column was fairly heavy reinforcement, but not abnormal. The frictional resistance obtained also showed great variation, but that was usual, and was presumably due to the difference in the length of the bars embedded: the lowest resistance was 243 lbs. per square inch of area of bar, and the highest was 614 lbs. But it was satisfactory to know that the 100 lbs. per square inch generally adopted allowed a considerable factor of safety in regard to slipping. He did not understand what Mr. Popplewell meant by "adhesion" and "frictional grip" on p. 24, where he said: "this is often called adhesion, but is more probably in the nature of a frictional grip preventing any relative motion of the surfaces of the steel and the concrete." He had always thought that adhesion was due to pressure, and he presumed grip caused pressure, and therefore he thought the adhesion was dependent on the grip.

Although the title of Mr. Burr's Paper was Composite Columns of Concrete and Steel, he noticed that in the text Mr. Burr spoke of the columns as reinforced-concrete columns. In Mr. Meik's opinion they were not proper reinforced-concrete columns, but were more of the nature of protected steel columns, such as had been used in America for many years for large buildings, and had been adopted in England in recent years. They consisted of a steel framing and what might be called a veneer of either concrete or ashlar; and a good example of this method of construction would be in existence in Great George Street shortly. The only advantage that could be claimed for it was, he believed, rapidity of

Mr. Meik. execution, and that was not always secured. In the two columns shown in *Figs. 1* (p. 42) there was no bond between the outer shell of concrete and the concrete inside the column; the outer shell was really useless, and would no doubt come off if the column were subjected to severe strains. Moreover, it was of no use as a protection against fire, and he thought Mr. Burr admitted that to be the case, because he said: "This outer concrete or mortar is simply a protecting shell, only the concrete lying within the steel being considered as load-carrying. This is necessary because in case of fire much of the outside or protecting material may be destroyed or knocked off, so that the carrying member should be considered to consist only of the steel reinforcing column and the concrete lying within it." Under those circumstances Mr. Burr put his column out of court as regarded resistance to fire. It was also out of court with regard to economical carrying-capacity. It was true that such steel columns would carry very heavy loads, but considered as reinforced concrete they were exceptionally expensive. Mr. Burr's columns contained $10\frac{1}{2}$ per cent. of steel, which was of course an extravagant allowance for a reinforced column. Even Mr. Popplewell's columns with 5 per cent. were rather heavily reinforced. Mr. Ellis had also used somewhat heavy reinforcement in the column which had to stand a load of 376 tons, namely, $6\frac{1}{2}$ per cent. If Mr. Burr had reduced his steel by one-half and doubled his concrete, he would have obtained a stronger and much cheaper column. From the few calculations he had made as to the cost in England Mr. Meik made out that there would be a saving of 30 per cent. by doing so. It was true the column would be of rather larger diameter, but unless the circumstances were exceptional that did not matter very much. After reading the Paper he had the impression that Mr. Burr was advocating the use of steel from the steelmaker's point of view, and assuming that to be the case, a reinforced column such as Mr. Burr put forward would be very advantageous; but from the user's point of view it would be very expensive, and he thought that to call it reinforced concrete and put it forward seriously as a favourable example was stretching a point too far.

Mr. Vawdrey. Mr. R. W. Vawdrey remarked that it was extremely gratifying to those members who were particularly interested in reinforced concrete that three such Papers should have been read at The Institution. He took it that the Authors of the experimental Papers would not claim they had produced any new results, but, as the President had remarked, it was of extreme importance that older conclusions should be confirmed, and the Papers did confirm the principles on which most engineers had been working. Taking first

Mr. Popplewell's Paper, he was not quite clear as to what the Author had really aimed at. How otherwise could the steel and concrete act in a column than together? Towards the end of the Paper the Author described tests of a further column by point loads at the end, and that seemed really to prove something, namely, that a point load which was not applied directly to the steel was yet transmitted to it by the concrete very quickly. But, assuming that the reinforced-concrete column was carrying the load, both materials must necessarily be compressed together, and as the compression was the same in each case, he failed to see how the testing of such a column tended in any way to cause a shifting of the steel in the concrete. In tension the case was quite different. In the tension bars at the bottom of a beam, for instance, the concrete would crack and fail long before the steel had reached its yield-point, and consequently there was a tendency to movement of the concrete along the steel; but where both were in compression, he failed to see that there was any tendency to movement until the actual crushing-load of the concrete was reached. On p. 24 Mr. Popplewell practically asked for a definition of adhesion. He did not quite understand what the Author meant by comparing it with frictional grip; presumably the two things were the same, whatever they were called. In connection with that point it was rather interesting to remark that the adhesion or grip on round bars appeared for some reason or other to be better than that on square bars. Possibly it was due to the contraction of the concrete around the bar. In some experiments he had made recently, on much the same lines as Mr. Popplewell's extraction experiments, it had been brought out fairly clearly that round bars, whether mechanical-bond bars or plain bars, were much superior to square bars, although the sectional area of the round bar was larger in proportion to its perimeter than that of the square bar, and the contrary effect would be expected from first principles. He noticed that Mr. Popplewell adopted the ordinary method of constructing columns, namely, timbering three sides and making up the fourth with separate pieces as the concrete was raised. That was by far the best method. He had tried both ways, and had found that the only case in which it was at all satisfactory to pour concrete from the top of the column was when the column-centering was big enough to allow a man to get down into the column. That was practicable only in columns not less than 3 or 4 feet in diameter. In such cases it could be done quite satisfactorily by having centering of a circular form and allowing a man to get down into the middle to place the concrete, a method which was adopted extensively in the United States.

Mr. Vawdrey.

Mr. Vawdrey. He did not quite understand the meaning of the paragraph dealing with column E on p. 32, where Mr. Popplewell said: "It might be suggested that the effect of the three first loadings and releases has been to disturb the grip." Assuming that disturbance of the grip between the steel and the concrete had taken place, he did not understand how it would throw more load on to the steel. It seemed that if both steel and concrete were being compressed to the same extent, there was no tendency in the steel to move relative to the concrete, and the grip, whether it existed or not, had no effect. It was rather a pity no test-block had been made from the concrete with which column A was mixed. It appeared to have differed considerably from the others in composition. He thought Mr. Popplewell was absolutely right in suggesting that the slight divergence of his curves was due to the fact that the movement or contraction of the concrete was necessarily measured at the outside of the column, whereas the steel was some little way inside. It was equally conceivable that there might be a weak part of the concrete which would compress at the outside edge of the column, or the reverse, while the portion of concrete actually next the steel would be carried down with the steel. He wished to congratulate Mr. Popplewell on the tests he had made on the extraction of bars from concrete, because a test so rarely acted exactly as it ought to do. The effect of the steel reaching its yield-point, and so pulling away from the concrete, was exactly what might be expected to happen. The Table of frictional resistance to slipping (p. 36) was extremely interesting, but Mr. Vawdrey would like to know the reason for the very marked discrepancy between the first three items and the next three. The average of the column headed "gripping force per unit area of bar" was 516 lbs. per square inch in the first three and 337 lbs. in the next three, and yet in the last three cases the age of the concrete was 7 months, as compared with 2 and $2\frac{1}{2}$ months in the first three. Surely it would have been expected that, after 7 months, the grip of the concrete on the bar would be appreciably greater than at 2 months. It appeared as though the grip of the concrete diminished with age. The point he had already referred to arose again at the bottom of that page, where Mr. Popplewell calculated a stress of 3·86 tons per square inch in the bars and said that it amounted to a load of about $1\frac{3}{4}$ ton on each bar, "which load, when compared with the figures in the last Table, was obviously nothing like sufficient to cause a movement of the concrete in the steel." But that stress in the steel was not tending in any way to move the bar in the concrete, and Mr. Vawdrey did not understand what point Mr. Popplewell was trying

to make. Both the steel and the concrete were stressed, and, of course, in direct proportion to their moduli of elasticity. Mr. Vawdrey.

Turning to Mr. Burr's Paper, he entirely agreed with Mr. Meik that the columns were not really reinforced concrete, but rather steel encased in concrete. At the bottom of p. 39 Mr. Burr said: "In columns of this type the reinforcement is not so distributed or held as to constitute a direct load-carrying element of the column." But surely that was not so. A purely reinforced-concrete column might occasionally have, as Mr. Meik had pointed out, as much as 10 or 12 per cent. of reinforcement, and he had known many such cases. Taking the ratio of the two moduli of elasticity as 15, considerably more than half the total load on the column was carried by the separate steel bars. Surely it was incorrect in such a case to say that the steel did not form a direct load-carrying element of the column. Farther on Mr. Burr said: "It has been shown by Professor A. N. Talbot and others that with this type of reinforcement, the concrete receives little support from the reinforcing steel until the load is nearly equal to the ultimate resistance of plain concrete." That was directly disproved by Mr. Popplewell's experiments, which showed that even from the very early stages of the test the steel did actually take its proportion of the load. He agreed with Mr. Burr that the frame type of column might be very suitable for extremely high buildings—say, forty stories—but that was not on account of its intrinsic merits as a reinforced column, but because it was capable of carrying a very considerable load before the concrete was put there, and the strength of the steel could be used from a structural point of view, which was a great advantage in many cases. The percentage of steel in the frame columns was much higher than could be used economically, and for such small columns it would be quite impossible to adopt that method with any economy. He dissented from the statement on p. 51 that the frame column was the best form for effective banding. A straight strap from one angle to the next was surely not an effective banding or binding of the concrete in between the vertical angles. It necessarily stiffened the steel, whether surrounded by concrete or not; but the effect of circumferential binding in the column—either spiral binding or circular straps—was also to confine the concrete, and that, he claimed, was not done at all well by straight straps as in the examples shown.

With regard to the works at Shanghai, the raft construction was very interesting and very suitable for that class of work. Occasionally it was advantageous to construct a slab on the top of the beams, care being taken, of course, to tie it down to the beams,

Mr. Vawdrey. because the ground-pressure tended to force the slab away from them. But, when that point had been attended to, it was often very advantageous to have the actual slab, which, of course, formed the tee of all the tee-beams, on the top of the work, so as to form the permanent floor. The question of designing the beams as tee-beams did not present much difficulty because they were acting as tee-beams at the centre of the span instead of the supports, or vice versa; they might act as tee-beams at the supports and as plain rectangular beams in the centre of the span. It did not make very much difference, from the point of view of economy, which way they were taken. The cover of the steel was mentioned as having been never less than 1 inch. That appeared to be rather too much for slabs. After all, with the small bars used in slabs, $\frac{1}{2}$ inch cover was quite sufficient for protection against fire, and 1 inch in beams or slabs merely had the effect of causing cracks, because the surface of the concrete was farther away from the neutral axis and consequently had to extend more under load. In columns, of course, the cover might be of any thickness. With regard to the costs, that of the piles was extremely low. He had never known any reinforced-concrete piles in England carried out as cheaply as 3*s.* 7$\frac{1}{2}$*d.* per cubic foot. On the other hand, the cost of the superstructure, especially in the wharf, appeared to be rather heavy, 3*s.* 6*d.* per cubic foot, verging on £5 per cubic yard. A more usual price in England for that class of work would be between £3 and £4 per cubic yard.

Mr. Faber. Mr. Oscar Faber, referring to Mr. Ellis's statement that the beams had been assumed to have partial continuity, and the bending-moments had been taken as $Wl/12$ at the centre of the span, observed that it would be interesting to know what had influenced Mr. Ellis to decide upon that value, because although it had been given as the bending-moment at the centre of the beam, it did not appear to have much theoretical or other justification. The bending-moment at the centre of a continuous reinforced-concrete beam depended not only on the loaded span, but also largely on the condition of the adjacent spans—whether they were loaded or not. The worst possible condition that could occur was for alternate bays to be loaded and the intermediate bays to be unloaded; and even then there would remain on the intermediate bays the dead load due to the weight of the floor and the beams. Taking that into account, with the loads given for these particular warehouses, namely, a live load of 500 lbs., and a dead load amounting to about 100 lbs., per square foot, exact calculations showed that the maximum bending-moment that could possibly occur under the worst possible conditions was $Wl/13{\cdot}1$, or

10 per cent. less than the value actually adopted. Any such calculation assumed that there was no settlement of the supports. In the building under consideration Mr. Ellis had no doubt been justified in making some allowance for settlement, but exactly what allowance ought to be made was obviously a moot point. The actual stress in any reinforced-concrete beam calculated as continuous was uncertain when settlement occurred. He would like to know whether the outside columns had been designed for eccentric loading or not, because it was not very clear from the drawing to what extent the outside columns differed from the inside columns. There appeared to be a considerable diversity of practice in regard to the extent to which outside columns should be designed for eccentric loading, and obviously this had a marked influence on the size of such columns. Mr. Faber.

He considered Mr. Popplewell's Paper a very valuable one. This seemed to be the first time that tests had been made to find the relative stresses on the steel and the concrete, except by finding experimentally the increase in the total load which the column would carry before breaking when longitudinal reinforcement was added. All previous determinations of the value attaching to the steel relatively to the concrete had been deduced from the ultimate loading on the column, and up to the present the value of m, the ratio between the moduli of elasticity of steel and concrete, had always been taken at a lower figure than the 19·8 which Mr. Popplewell had found. The value most frequently adopted was 15; in some recent reports it was even lower, e.g., 10 or 8. The reason for that would appear to be that in all column-tests made up to the present the ends of the columns had not been enlarged, as Mr. Popplewell had made them: the specimens had been so designed that the steel was unable to take its fair share of load near the ends, failure therefore occurring at the ends. It was impossible for the steel at the ends to be taking its fair share of the stress, since that stress was transmitted to it only by adhesion; and therefore, in order to secure equal strength of the column at this point, it was essential that the column should be gradually enlarged towards the ends in such a way as to enable the concrete alone to take the whole load at this point. Nearly all the tests made hitherto, which had given a comparatively low value for the steel in reinforced columns, had been largely vitiated by that point having been overlooked. Mr. Popplewell, by enlarging the ends of his columns, and so preventing them from failing at their weakest point, had found that the steel was much more effective than had been thought hitherto. That was particularly interesting, because a very low value

Mr. Faber. was usually assumed for steel, and there were many reasons for supposing that its real value was much higher. For instance, beams reinforced heavily on the compression side, had always shown much greater strength in compression than that deduced from the ordinary formula, taking the ratio between the moduli of elasticity of the two materials as 15. Actual experiments had shown that the strength of steel in compression, to which, of course, the stress was transmitted solely by adhesion, had been always much higher than that given by the usual formula; and that conclusion was borne out by Mr. Popplewell's valuable experiments, which were likely to justify giving to the steel in columns a higher value than hitherto. Mr. Burr's columns, like most other reinforced-concrete columns, had not had their ends enlarged during testing, and therefore there had been difficulty in knowing to what extent the steel was enabled to take its load before some crushing of the concrete occurred. The results might have given even a higher value for the steel if the ends had been enlarged in such a way as to ensure the two materials acting together. The value of that particular type of column would appear to be very considerable in buildings of many stories where the size of the column had to be kept within certain limits, and where there was no practical alternative to the use of a high percentage of steel, although it was known to be much more expensive.

Mr. Smith. Mr. Harold D. Smith considered that, in view of the very bad ground in which the groups of piles rested, greater precautions might have been taken by Mr. Ellis to avoid settlement, which would lead to the breaking-up of the floor in a longitudinal direction. In the cross section the bottom walings connecting the groups of piles and the top floor-beam were connected by diagonals, which made the whole structure between the bottom waling and the floor into a stiff girder about 12 feet deep; but in the longitudinal section diagonals were only provided to the end one or two bays, and if one small group of piles for one pier should settle slightly, the floor would have an undue stress thrown upon it and would not be stiff enough to distribute the load over the adjoining groups of piles. In that event the floor might be damaged. With a deeper girder, formed by the introduction of a continuous system of diagonals, the load might have been distributed efficiently over other groups and the structure made safer. Each warehouse was another example of the same thing; the whole of the distribution was taken in the bottom slab and the warehouse itself was a skeleton structure, with walls consisting of brick panels, which could not augment its general stability. He thought that if

diagonals had been introduced in the walls, so that the floors and columns might form members of a girder as deep as the building, it would have been much stiffer and much more economical than a girder of the depth of the foundation-slab. That seemed to be particularly important on a foundation which allowed a building to settle 14 inches within 12 months of its construction. Diagonals might also have been introduced between the columns and floors where possible, and the load spread over the floor, and that also would have resulted, he thought, in economy in the design of the floor. The floor described in the Paper was capable of carrying the weight over three adjacent bays. If diagonals had been introduced the floor might have been made capable of distributing the load over one bay only. In the inside, between the columns, diagonals could not have been introduced everywhere, but there seemed to be no reason why, where diagonals could not be introduced in the bottom floor, they should not step up the next floor three bays over the one bay gap and so produce a continuous girder. In temporary wooden buildings, 20 feet by 20 feet, used as offices or stores while alterations were going on, if hoop-iron diagonals were placed between the upright posts, and nailed to each post, the building could be picked up by the corner or balanced on two corners, and it would hold together remarkably well without distortion or damage. At the present time the Great Western Railway Company were reconstructing a wharf built at Plymouth about 30 years ago by the late Sir James Inglis. The greenheart and jarrah piles had been very much diminished in section by the action of marine worms, and the white-wood bracings and walings had disappeared entirely. The wharf was 820 feet long by 30 feet wide. Various methods of reconstruction were considered, and ultimately reinforced concrete was adopted for the following rather important reasons:—The cost would be about two-thirds of the cost of rebuilding in timber. Reinforced concrete was not attacked by worms. The work could be carried out in reinforced concrete with very little disturbance to the business of the wharf, which could proceed as usual, except during the reconstruction of the floor itself. The existing wharf formed a scaffold for the pile-driver, which constituted the only occupation of the wharf by temporary structures. The main beams, bracings, walings, and girders could all be constructed under the existing floor without disturbing the traffic until it was necessary to build the floor itself. The foundation was rock, or what was considered to be rock, in every case. The piles were hooped like the columns described in Mr. Ellis's Paper, and had been designed by the Considère Company. It had been

Mr. Smith.

Mr. Smith. possible to drive the piles about 2 or 3 feet deeper than the depth at which the boring-tackle was brought up by the rock. They had been driven without any helmet or protection, but a wooden dolly had always been used. Helmets had been introduced lately to save the dollies. An 18-inch greenheart dolly driving 15-inch piles was knocked to pieces after it had driven five piles, but now, with a cast-iron helmet and sawdust, a dolly would drive more than twenty piles. The wharf carried four lines of sidings, a crane-road carrying a hydraulic crane weighing 60 tons, and a live load of 4 cwt. per square foot. At first, in calculating bending-moments for the continuous joists and the continuous main cross girders, which rested on four piles and the wharf-wall, formulas were used such as were described in the text-books and mentioned in Mr. Ellis's Paper, but it was found that the distributed live load, the varying rolling load, and the dead load could not be taken into consideration without full calculations, which were made by Professor Claxton Fidler's graphical method for every possible combination of loading. The worst cases were taken, an envelope was drawn round the whole of them, and the girders and joists were designed to resist the corresponding bending-moment. He thought Mr. Ellis deserved congratulations on the success of his work; it was the most daring construction of a wharf that he had ever heard of.

Mr. Marsh. Mr. C. F. Marsh thought all three Authors were to be congratulated on having produced such interesting matter. With regard to Mr. Ellis's Paper, it appeared to him that the raft formation adopted at Shanghai was the only solution of the difficulty. He had seen lately a design for a similar foundation using inverted arches instead of straight beams, and it had struck him as being rather a good idea. Mr. Ellis stated that he took the stress on the steel at fifteen times the stress on the concrete, with an ultimate limit of 12,000 lbs. per square inch. Mr. Marsh did not quite see why he limited it in that way. It was very probable that the maximum stresses in the actual structure were about 1,000 lbs. per square inch on the concrete, and 15,000 lbs. per square inch on the steel, which were in the ratio of the moduli of elasticity. In the Table of the tests on the wharf-beam (p. 17) the factor of safety was given as $7\frac{2}{3}$, reckoned on the calculated super-load. That was taken on the 90 tons at which the beam failed, but it should have been taken on the 45 tons at which the first crack appeared, in which case it became $3\frac{5}{6}$ the calculated load. On the next page there was a Table in the second column of which it was shown that 1 : 2 : 4 concrete gave a higher result than 1 : $1\frac{1}{2}$: 3 concrete. That was curious, because in the third column, " Final Load," the reverse was

the case; and the reverse was also the case in the following Table (p. 19), showing the results of tests on unreinforced columns. He found on calculating the reinforced columns by the formulas given in the second Report of the Joint Committee of the Royal Institute of British Architects, that the safe load on the columns of 1 : $1\frac{1}{2}$: 3 concrete was 44·64 tons, and on those of 1 : 2 : 4 concrete 38·17 tons. The stress in the concrete in the first case was 1,002 lbs. per square inch, and on the 1 : 2 : 4 concrete 859 lbs. per square inch. Mr. Marsh.

With regard to Mr. Popplewell's Paper, there was a diagram on p. 25 which showed a gauge-length of 8 inches, but on p. 33 the length was given as 12 inches. From the calculations he had made he thought the latter must be the right lengths. With regard to the conclusions arrived at, he thought Mr. Popplewell had gone to work in the wrong way. *Fig. 5* gave the mean deformations of the concrete and the steel. The modulus of elasticity of the steel was very fairly known, and it was probable that the figure given was correct; but the modulus of elasticity of the concrete was not so certain. However, from the results of the experiments it was possible to get at the exact modulus of elasticity of the concrete. Taking the unit strain on the steel, calculating from it the stress on the steel, and multiplying that by the sectional area, the total stress on the steel was obtained; deducting it from the total pressure on the column, the stress on the concrete was found; and from that the modulus of elasticity of the concrete could be calculated, using the strain obtained from *Fig. 5*. Calculating in this manner, the results given in the Table on p. 66 had been obtained. It would be seen that the mean value for the ratio of the moduli of elasticity was 14·5, which was very near the usual value of 15.

In Mr. Burr's Paper a statement was made that longitudinal steel rods in columns might be capable of resisting by themselves some compression, but it was of such small amount as to exert little or no direct material influence upon the carrying-capacity of the member. Of course, concrete properly hooped could be stressed considerably higher than the usual allowance of 600 lbs. per square inch, and with sufficient hooping it was possible to get 1,000 lbs. per square inch, or more, on the concrete, and therefore mild steel could be stressed up to 15,000 or 16,000 lbs. per square inch, which was its working-capacity. Later on, Mr. Burr said: "It has been shown by Professor A. N. Talbot and others that with this type of reinforcement the concrete receives little support from the reinforcing steel until the load is nearly equal to the ultimate resistance of plain concrete." Although Professor Talbot

VALUES OF UNIT STRESSES IN STEEL AND CONCRETE AND OF MODULI OF ELASTICITY OF CONCRETE, CALCULATED FROM THE STRAINS RECORDED IN MR. POPPLEWELL'S PAPER.

	STEEL.						CONCRETE.						
	1	2	3	4	5	6	7	8	9	10	11	12	13
	Unit Strain (from Fig. 5) measured strain / 12	Modulus of Elasticity E_s (given).	Unit Stress 1 × 2	Sectional Area (given).	Total Stress 3 × 4	Total Pressure on Column.	Total Stress 6 − 5	Sectional Area (given).	Unit Stress 7 ÷ 8	Unit Strain (from Fig. 5) measured strain / 12	Modulus of Elasticity E_c 9 ÷ 10	$\frac{E_s}{E_c}$ or 2 ÷ 11	Mean Value of $\frac{E_s}{E_c}$
	Ins.		Lbs. per Sq. In.	Sq. Ins.	Lbs.	Lbs.	Lbs.	Sq. Ins.	Lbs. per Sq. In.				
Column A	0·00027	$30·2 \times 10^6$	8,154	1·77	14,432	30,240	15,808	34·23	462	0·00027	$1·711 \times 10^6$	17·65	14·51
Column B	0·00018	,,	5,436	,,	9,622	,,	20,618	,,	602	0·00021	$2·876 \times 10^6$	10·53	
Column D	0·0002	,,	6,040	,,	10,691	,,	19,549	,,	571	0·0002	$2·855 \times 10^6$	10·58	
Column E	0·00028	,,	8,456	,,	14,967	,,	15,273	,,	446	0·00027	$1·652 \times 10^6$	18·28	

and others had found that the hooping did not come into play until the stress on the concrete had reached a point at which unreinforced concrete would fail, the fact that the failure of the concrete was prevented allowed a higher value to be used for the working-stress. Mr. Marsh thought that some method should be adopted whereby a higher stress on the concrete in this type of column could be allowed, as suggested by Mr. Burr. He did not quite agree with other speakers that Mr. Burr's form of column was not an example of efficient hooping, because he considered that in a column of that nature, particularly the diagonally-braced column, the vertical reinforcement acted both as hooping and as longitudinal reinforcement. It certainly did hoop the concrete efficiently, and he believed that for such a column a safe stress on the concrete of 700 lbs. or 750 lbs. per square inch might be allowed, or even more. It was rather a pity that Mr. Burr had not included tests on reinforced-concrete columns hooped in the usual manner, for comparison. It would have been extremely interesting if he had tried some of those columns, with longitudinals approaching the same sectional area as in the columns tested. Mr. Marsh.

Mr. J. B. Ball stated that he had lately been engaged in carrying out reinforced-concrete structures of various kinds, from bridges to buildings, and in particular had put down recently a large raft on the banks of the Humber to form the foundation and pits for an engine-shed. It was somewhat similar to the raft adopted at Shanghai, in that on the Humber site there was about 25 to 30 feet of warp before anything solid was reached, and piling for the pits of the engine-shed would have cost a very considerable sum. After careful consideration, it was decided to put down a reinforced-concrete raft. Naturally, the design of an engine-shed floor, with pits about 3 feet deep, lent itself to the formation of a raft of a peculiarly strong character. The shed was 360 feet long, and 160 feet wide, and its superficial area was thus nearly 60,000 square feet. There were twelve pits, forming a series of corrugations, as it were, running throughout the length. The raft had been built only quite recently, and had not been fully tested, but a heavy locomotive had been running over the floor, and the results so far had been very satisfactory. The price of the reinforced concrete worked out at 1*s.* 9*d.* per cubic foot for the floors, foundations, and pits, or 3*s.* per square foot, or £2 1*s.* 7*d.* per lineal foot of the pits. Compared with ordinary construction in brick on a pile foundation, the reinforced-concrete structure—which he believed was unique in England, in that it had no wood of any kind for the attachment of chairs for carrying the rails in the shed, Mr. Ball.

Mr. Ball. and the top of the reinforced concrete formed the floor of the shed as well—had cost about £9,500, while the lowest estimate (a firm tender) for a brick design was £12,900. In his opinion that showed the desirability of going more fully into the question of the use of reinforced concrete for engineering structures. He had heard all kinds of views about this new form of construction, and a good deal of criticism and ridicule had been thrown upon it, but he had come to the conclusion that it had come to stay, and would have to be seriously taken into account where cost was an important consideration. He did not believe that it was suitable for all kinds of structures: it had its limits; but it had also its uses, and, from the experience he had had, extending now over about 7 years, he was beginning to believe still more in it. In addition to the concrete raft, he was also carrying out a large granary, about 80 feet high, in reinforced concrete from top to bottom. It had been put out to competition based on general drawings supplied by his company, and various experts in reinforced concrete had submitted designs. These had been carefully analysed, and the differences between the formulas used by the several experts had been found to be remarkable. For instance, some had taken the bending-moment at the centre of a beam as $Wl/24$, others as $Wl/10$. The result was that the cubic contents of the concrete and steel in the building, which had to be stated in the tender, varied very considerably, the quantities ranging from 350 to 380 tons of steel reinforcement, with concrete in proportion. In the design actually accepted, and now, he believed, being very successfully carried out, there was about 500 tons of steel —which involved a fairly substantial increase. There were reinforced-concrete piles on the Humber which were sustaining 25 to 50 tons per pile. The cost of these varied from 5*s.* to 6*s.* 9*d.* per cubic foot. One curious incident had occurred in connection with the comparative skin-friction of timber and reinforced-concrete piles. Two piles were left standing for about a fortnight, and when driving was started again a considerable number of blows were given before there was any sign of yielding on the part of the timber pile, whereas the ferro-concrete pile sank at the first blow exactly as it had been sinking a fortnight before. Whether others had had the same experience of reinforced-concrete piles he did not know, but he proposed to make some further experiments on the subject. The ratio of steel to concrete in the granary as a whole, including silos, worked out at 1·43 per cent., there being 166,050 cubic feet of concrete and 500 tons of steel. The cost was approximately 2*s.* 9½*d.* per cubic foot of concrete, or about 3·4*d.* per cubic foot of building, as compared with the 3*s.* and 5·17*d.* given in Mr. Ellis's Paper. The cross-sectional percentage of steel in the columns varied

from 5·28 per cent. on the bottom floor to 1·22 per cent. on the top floor, the columns being reinforced with round bars tied together at intervals but not hooped spirally, the maximum load being about 300 tons, as compared with percentages ranging from 5·70 to 2·2 in the warehouse at Shanghai, the maximum load there being 376 tons, and the columns carrying this load being hooped. The granary was supported on timber piles, driven in groups in positions underneath the columns of the building. The heads of the piles were surrounded with concrete bound together with Clinton wire mesh. Referring to the Shanghai wharf, the cross-sectional ratio of steel to concrete in the main and secondary beams appeared to be about 2·8 and 1·35 per cent. respectively. He had had the percentages taken out in the case of the main beams of several bridges he had erected recently, and he found these to range from 3 per cent. in a girder 32 feet long, ratio of depth to span 1 : 15, to as much as 8·7 per cent. in a girder 80 feet long, ratio of depth to span 1 : 9; this latter girder was, however, very heavily loaded, being one of two main girders carrying a 36-foot main road. There appeared to be no doubt that, generally speaking, structures in reinforced concrete were less costly than those built in brick and steel. He had in mind bridges where carrying out the work in reinforced concrete had effected a saving of 15 to 30 per cent. on the estimated cost of a brick and steel structure. This, together with the reduction in the cost of maintenance—which was always necessary in the case of a steel structure—rendered the adoption of such structures worthy of consideration. Mr. Ellis referred to the cracking across several bays of the wharf. Mr. Ball had had several instances in which slight cracks, not at all deep, had occurred near the ends of bridge-girders, taking up a direction along the lines of shear. In one bridge, built on very bad foundations, the weight of the bank tipped behind the abutments forced up the ground between them, fracturing the cross ties which tied the heads of the foundation together, sluing the bridge bodily round nearly 2 feet and cracking the girders in a number of places. The damaged work was repaired, and a concrete invert was built between the wings and abutments; since then no further trouble had arisen. Although water had got through where cracks appeared, there was no sign of rust on the faces, showing that there was no corrosion. The valuable monolithic nature of the work was well illustrated by this example. Mr. Ball.

Mr. Popplewell's Paper gave information of considerable interest and value, the question of the adhesion between steel and concrete in reinforced work being one which had given rise to considerable

Mr. Ball. discussion, and had resulted in bars of various forms and sections being invented with a view to overcome what was thought to be a somewhat unsatisfactory state of things in the case of ordinary round or square bars embedded in concrete. It would, he thought, have been advantageous if Mr. Popplewell had extended his experiments and investigated what additional frictional resistance or adhesion was obtained where such special forms of bars were adopted. Valuable as such experiments were, when the results had to be applied to the design of actual works, Mr. Ball felt that they should be considerably discounted. In the first place, the conditions under which experimental work was carried out, and those obtaining in actual practice were very different. For example, Mr. Popplewell said, "The moulds were left in the vertical position and allowed to remain in a somewhat damp atmosphere for 14 days; after which, the boarding having been removed, the columns were kept in a damp position until ready for testing. During this last period they were frequently sprinkled with water." In actual practice there were continual changes of atmospheric conditions—varying from dry to damp, and from hot to cold and frost—and, although precautions might be stipulated, they were often imperfectly taken. The mixings of large quantities of concrete were also subject to variations which would not exist in the case of the small quantities required for experimental work: so that although tests made on concrete used in an experimental column might fairly indicate its strength in such a column, the result might be altogether too high or too low for portions of similar concrete used in different parts of a large structure. This was clearly shown by the diversity in the results obtained from samples taken from different mixings used on works; so that an experimental ratio of the moduli of elasticity of 19·8, such as that given in Mr. Popplewell's Paper, varying as it did from the accepted value of 15 (whilst, no doubt, correct for the particular columns experimented upon), ought not, Mr. Ball thought, to be used in calculating columns for actual structures, even though a similar concrete were used. The success of structures in reinforced concrete depended very largely upon the care exercised in carrying out the work; and very careful supervision was necessary in the mixing of the concrete, the correct disposition of the reinforcing bars, the actual placing of the concrete in position, the stability of shuttering, etc. These matters admitted of so much more perfection in experiments than in actual practice that he again ventured to emphasize the point that a considerable margin of safety should be allowed in applying the results of such experiments to practical work.

Mr. E. F. Etchells wished to draw attention to the method of failure of the pillars in each of the three Papers. Mr. Ellis's reinforced-concrete pillars were nine diameters in length and they failed by general flexure. It was an exceptional thing to have a pillar of only nine diameters fail by bending. The plain concrete pillars of the same length failed by direct compression, or, more precisely, by shearing along diagonal planes, giving the usual conical fracture. Mr. Burr's pillars were generally of fourteen diameters, and three out of the four of the mild-steel pillars failed by flexure, but none of the reinforced-concrete columns failed in that manner; these failed by bulging due to the bending of the steelwork locally. Mr. Popplewell gave the ratio of length to diameter for his columns as 16, calculated on the maximum diameter, and stated that he considered them short, and therefore failure by bending need not be considered. On examining his diagrams, in which the compressions were given at each of four points, it would be seen they were so regular that his assumption was fully justified. In the French Government Regulations permission was given to go up to twenty diameters, taking only direct compression; and it would be a serious matter if government regulations allowed up to twenty diameters while actual experiments showed that the pillars failed by bending at nine diameters. Mr. Ellis's results were so low as to suggest that there had been some eccentricity of loading to cause bending: such low figures as he recorded were not in accord with the general run of experimental results. On the other hand, Mr. Popplewell's tests had probably been carried out with very great care under laboratory conditions. Perhaps, therefore, Mr. Burr's experiments might be taken as coming fairly between the other two, and as representing the conditions which would be found in a general case. Referring for a moment to the type of Mr. Burr's pillar, it had been said that it was doubtful whether it was really reinforced concrete; it was quite as much reinforced steelwork. If engineers were free now to consider that it was reinforced concrete, it meant that another barrier had been broken down. For some time past pillars had been divided into two classes, those with straight-line hooping or laterals, and those known as hooped pillars. As the only detailed report published in England had broken down that barrier and allowed the two classes of pillars to merge one into the other, it would appear that reinforced-concrete pillars might be allowed to include the new type, although the reinforcement was excessive, and the cost, he should imagine, prohibitive for universal adoption. There was also another objection to that type of pillar,

Mr. Etchells.

Mr. Etchells. inasmuch as it was very difficult to fill in the concrete right up to the top so that the concrete inside at the joints of the beams and girders would be fully capable of taking the stress which the calculations showed. Of course, if there was a cavity at the top the concrete below did not get a chance to take the load; it only prevented the bulging of the steelwork. Mr. Burr gave the stresses on the angle pillars at failure as $16\frac{1}{2}$ and 17 tons, and the ratio l/r as 34. The result was entirely anomalous, because on the channel pillar, for which the ratio l/r was less, the breaking load was less instead of being greater. Mr. Burr went on to say: "From this it will be seen that Nos. 1 and 2—the angle-bar columns with the usual lattice bracing—show ultimate resistances at failure distinctly higher than Nos. 5 and 6—the channel-bar columns without positive bracing between them, but with battens only. The advantage of holding the members of a column under compression by complete effective bracing is thus apparent." Mr. Etchells dissented from that entirely. Considering the individual elements of the pillar, it would be found that the ratio l/r was 25 in the first case, with an average breaking-stress of $16\frac{3}{4}$ tons, and in the case of the channels, taking the length of the channels between the bindings, the ratio was 50, thus accounting for the lower crippling stress of about $14\frac{1}{2}$ tons. What Mr. Burr had shown, however, was the advantage of hooping the pillar. Mr. Etchells quite agreed that generally the braced pillar was stronger than the pillar with merely battens or plates. Professor Müller-Breslau, in a Government inquiry in Germany, had found the same action occurring. In that case it was not a laboratory experiment, but a test of a full-sized pillar of a gasometer; and unfortunately, it was not an intentional test, but an accident in which thirteen men were killed and forty-seven injured. That accident was attributed largely to unbraced pillars having unequal loading on the sides. On p. 28 Mr. Popplewell gave the yield-point of the steel as 18 tons, and the limit of elasticity as 12·5 tons. A difference of $5\frac{1}{2}$ tons on ordinary mild structural steel certainly seemed considerable, if not exceptional. Since proposals had been put forward to base the working-stress on the elastic limit, and also proposals to take one-half the yield-point, Mr. Etchells submitted that in the present case it made a great difference whether the half of 18 was to be taken, or the half of 12. The Table on p. 36 showed a yield-point ranging from 7·5 to 7·96 tons, and Mr. Etchells would like to ask whether that yield-point, which was about 8 tons, was the same yield-point as the one mentioned on a previous page.

Mr. F. E. Wentworth-Sheilds considered it a great advantage to The Institution to have three such Papers before it, as they gave rise to very interesting comparisons. After all, in reinforced concrete the great question to a practical engineer was whether the formulas he used were dependable and whether the assumptions made in deriving those formulas were correct; and, if not, how far would the fact that those assumptions were not correct affect the results, by making the structures either unsafe or too extravagant. The strength of the reinforced-concrete columns tested by Mr. Popplewell could be calculated by the ordinary rules, such as those laid down by the Royal Institute of British Architects; and assuming that the concrete would bear safely 500 lbs. per square inch, and that m, the ratio of the modulus of elasticity of the concrete to that of the steel, had a value of 15, the column would bear safely a load of about 13½ tons. A great deal of thought had been given to the question whether it was right to assume, as the R.I.B.A. report did, that m had a value of 15. It was interesting to notice that, although values for m had been suggested that evening ranging from 20 to 10, the difference in the safe strength of the column was not varied very much by taking those different values. For instance, if m were assumed to have a value of 20, the safe load on Mr. Popplewell's columns would be 15½ tons; while if it were 10, the safe load would be 11½ tons. Even with that very wide variation of 100 per cent. in the value of m, therefore, there was only a variation in the calculated safe strength from 11½ to 15½. In all cases there was a substantial factor of safety, and he thought there need be no great anxiety if the mean of those three values, namely 15, was taken for m. In such a column as the composite column described by Mr. Burr the difference that arose from assuming various values of m became considerably more marked. For instance, in the design *a* (*Figs. 1*, p. 42) if m was assumed to be 20, the safe load—again assuming the safe stress on concrete to be 500 lbs. per square inch—worked out at just over 29 tons; when m was taken as 15, the safe load worked out at 24 tons; and when m was 10, it worked out at 18·7 tons. On account of the large proportion of steel a fairly wide range of values for the safe strength of the column was found. It was satisfactory to notice, however, that even with a composite column of this type the ordinary rules gave a safe strength which was one-fourth, or less, of the ultimate load. Another point much debated by users of reinforced concrete was the bending-moment assumed for calculations in the case of a continuous reinforced-concrete beam. It was customary, and no doubt wise, to assume that the bending-moment was rather greater than theory would suggest, owing to the fear that settlement might take place,

Mr. Wentworth-Sheilds.

Mr. Wentworth-Sheilds.

or that the continuity might be imperfect in some way. To his mind that was really one of the points on which practical men had need to be most cautious, and with regard to which ignorance on the subject of reinforced concrete was greatest. Engineers did not really know what practical conditions were necessary to ensure perfect continuity in beams. It was supposed that if the bars overlapped the point of support to some extent, say, 80 diameters, or according to the rules laid down now by the London County Council—there would be perfect continuity, that was, that the tension on one bar would be transmitted through the concrete to the adjoining bar. It was supposed that this was so on account of certain experiments which had been made to show the intensity of the frictional grip between concrete and steel; but he thought he was right in saying that no satisfactory experiments had been made to show the frictional grip between concrete and steel under the circumstances in which they were placed in the case of continuous beams, namely with bars overlapping one another and lying side by side. He hoped that when the Institution Committee on Reinforced Concrete had finished the work they were now engaged upon, they would turn their attention to that matter and carry out some experiments which would allow engineers to speak with more certainty as to what was the proper design in order to ensure continuity in continuous beams. With regard to the wharves at Shanghai, he thought Mr. Ellis was an exceedingly bold man. He had put a reinforced-concrete platform on groups of four piles, which were calculated to stand a load of 45 tons; he had loaded those piles with 45 tons and found them to go down; none the less his sleep was undisturbed. For himself Mr. Sheilds confessed he would be a little uneasy if he had a platform on piles which were known to be liable to settle under the maximum working-load; though it was much easier to say that than to suggest something better. He was filled with admiration for the way in which Mr. Ellis had overcome the very difficult problem of placing a safe wharf on soft silt. In the case of the wharf-wall, the top of which was shown in Fig. 9, Plate 2, Mr. Ellis said that he put the backing behind it before the ties which secured the wall back to the foundations of the heavy building behind were put in, and consequently the wall, not being stable enough, moved forward slightly. He then removed the backing and the wall came back to its original position. Mr. Sheilds had noticed the same thing in the case of retaining-walls, which moved forward slightly during a very low tide, when the water-pressure was small, and moved back into place when the tide came in again. It rather looked as if the supporting earth sustained elastic deformation and was able to some extent to regain

its former shape and replace the wall. He thought all the members would envy and admire the very low unit cost of the work described. He believed very few engineers would be able to carry out reinforced-concrete work for 3*s.* 7*d.* per cubic foot. The low cost was partly explained by the fact that unskilled labour in Shanghai cost 10*d.* to 1*s.* per day, but it would be interesting to have the cost of the cement and the steel on the site, from which it would be easy to make some rough analysis of the total costs. Mr. Went-worth-Sheilds.

Mr. Maurice Fitzmaurice desired to say how much he appreciated Mr. Ellis's Paper, because the particulars given, such as the information with regard to settlement, the methods of carrying out the work, and the costs, were exactly the information required by many members of The Institution, and certainly what he wished to have himself. He thought The Institution had been very fortunate during the last two or three years in getting a number of its American members to bring forward Papers, and it was indebted very much to Mr. Burr, who was a prominent engineer in New York, for giving the information about columns that had been tested at the great Columbia University, where he was Professor of Civil Engineering. He was sorry that Mr. Meik and others had rather criticized some minor points relating to the columns. He did not think it mattered whether they were called reinforced concrete or reinforced steel, because the members knew exactly what was being spoken about; the Paper gave illustrations of the columns and exact details of their construction, and stated the quantities of concrete and steel that were being dealt with. The question whether such columns were economical or not depended, of course, entirely on where they were built. These columns appeared to take up the minimum of space, and where land was almost unobtainable at any price that was a very important matter. Some people seemed to forget that in New York, where such columns were used, the city was growing upwards, and not sideways, and it was absolutely necessary to have columns which could be built quickly, took up little space, and were capable of carrying very heavy loads. When it was remembered that twenty stories was there considered an ordinary height for a building, he thought nobody would think of using what was generally called in England reinforced concrete for the main columns if they desired to get such a building done quickly. Therefore the members were indebted to Mr. Burr for giving The Institution the information, which he might have given to some of the many engineering institutions in America. He wished to make some remarks on the conduct of experiments and tests in connection with constructional work. For some time he had been taking note of the experimental Mr. Maurice Fitzmaurice.

Mr. Maurice Fitzmaurice. Papers presented to The Institution. Unfortunately there were not many, but there had been a few, and some of them were of great value; others, however, in his opinion, were worthless—although, of course, there might be two opinions on that point. He could not help comparing the three Papers under discussion with others which The Institution had received recently, and which, as far as he could see, did not add much to existing knowledge, and certainly did nothing to influence the current practice of engineers. The present Papers were of a totally different character. They caused everyone to think carefully about reinforced-concrete work. There were so many advocates of different ways of dealing with reinforced-concrete construction, and so many theories based on very little experience, that the information in the Papers before The Institution might make engineers reflect whether they knew as much about reinforced concrete as some people supposed. The moral of the Papers under discussion and of those he had heard recently—if a moral could be drawn from them—appeared to be that, as far as possible, experiments of the kind referred to—in connection with constructional work—ought to be carried out by engineers dealing with that class of work. If the experiments were not carried out by engineers who were actually dealing with such work, the experimenters, if they had had no practical experience, ought to consult, to a certain extent, men who had practical experience, so as to be sure that the experiments justified the expenditure on them and that the results likely to be obtained from them would be useful to engineers. He was not one who objected to spending money on experiments. On the contrary, he would always advise The Institution, and any other body he might be connected with, to spend money freely on experiments. He would go farther and say that, when The Institution had got over the financial effort of constructing its new building, he would be very pleased if it were to set aside annually a sum of money to be given to selected resident engineers on large works who were in a position to make experiments which might yield useful results, and who would be able to do work which would give valuable information to the members. If there was one thing he detested it was seeing money spent on experiments which were quite useless to members of The Institution or to anybody else.

Mr. Read. Mr. R. J. G. Read remarked that it was very satisfactory to find that an assumption always made in designing columns and other members in reinforced concrete was verified by Mr. Popplewell's experiments. One thing that required further investigation was the amount of concrete that must be connected with the steel in

order to transmit the pressures. He had often noticed a number of reinforcing rods bundled together so as to leave little room for concrete between them, and it seemed to him doubtful whether the full value of the steel could be utilized under those circumstances. Mr. Popplewell had measured his compressions to $\frac{1}{1,000}$ inch on a length of 12 inches, and it would be interesting to know if there was any difference in the nature of the result when the compression was measured over a longer length. Mr. Burr had measured his results to $\frac{1}{10,000}$ inch on a length of 70 inches. The result of Mr. Burr's tests of the relative strength of columns made of steel alone and of steel filled with concrete was about what engineers had been accustomed to accept, namely, that the composite columns would bear about 50 per cent. more load than the plain steel. Mr. Read would like to know how much of that extra strength was relied upon in the calculations for the columns in the high buildings in New York referred to by Mr. Burr. Engineers were accustomed to use in the steelwork of buildings riveted stanchions and columns of various forms, which were filled with concrete, but the extra strength due to the concrete was seldom counted upon in the design of the columns. In the columns of the new building of The Institution sections of various forms would be filled with concrete, and no doubt they would be very much stiffened; but that extra stiffening had not been taken into account in calculating the strength of the steelwork, which was sufficient in itself. Comparing the results arrived at by Messrs. Popplewell and Burr with regard to the compression of the columns, he found that Mr. Popplewell compressed a true reinforced-concrete column $\frac{14}{1,000}$ inch in 12 inches, while Mr. Burr compressed plain steel columns $\frac{64}{1,000}$ inch in 70 inches. If the proportions were the same, that would be about $\frac{11}{1,000}$ inch in 12 inches, while the composite columns were only compressed $\frac{6}{1,000}$ inch in 12 inches, so that Mr. Burr's columns apparently did not shorten as much under the load as those experimented upon by Mr. Popplewell. In calculating these strains he had taken the limit of load mentioned by Mr. Popplewell (50 tons). The columns were not broken, but the compressions under that load were given, and similar data could be obtained from the curves given by Mr. Burr. He did not think the comparison was quite fair in regard to the stiffness of reinforced-concrete—in the usual acceptation of the term—and composite columns. In the reinforced-concrete column the resistance of the steel might be taken in the usual way as fifteen times that of the concrete for equal areas. Comparing the columns in that way, the equivalent concrete section of Mr. Popple-

Mr. Read.

Mr. Read. well's column would be 60·6 square inches, while Mr. Burr's steel columns would be 60 and 71 square inches without any concrete, and with concrete 92 square inches. Therefore the latter had a more effective area, and the compression consequently did not show as much.

With regard to the Paper on the Shanghai wharves, he thought that the piles which were lengthened and re-driven might well have been left as they were, considering that it took eighty blows to start them. They must have been thoroughly set; and as the working-load on each of the four piles was only about 11 tons, it would appear that they were quite capable of supporting their load. The trouble in lengthening piles was the delay, the piles having to be left about 6 weeks before they could be re-driven. It was often very difficult to tell exactly what length of pile was required for a given site. In one instance in his own experience he had a sample pile made and driven as far as it would go. The remaining piles were then made of the length thus indicated, but not one of them stopped at the same depth: the sample pile had gone down into the only hard piece of ground on the site. The other piles had to be driven 8 to 10 feet below the surface of the ground, but as they carried columns it was possible to excavate down and build up, and thus carry on the work without much delay. He noticed that the surface of the concrete slab under the warehouse at Shanghai was at the bottom and the stiffening beams formed ribs above. That arrangement had the advantage that it allowed one to see what was being done. When the concrete had set and the shuttering was cleared away the beams were visible. On the other hand, it was often more economical to turn the slab the opposite way, leaving the flat surface to form the floor of the warehouse, in which case the stiffening ribs were underneath and inaccessible. He had found it a good plan in soft ground to cover the surface with dry brick rubbish rammed well down, instead of using a layer of lime concrete as at Shanghai. The brick rubbish absorbed moisture. The channels for the beams were made by building rough trenches in brickwork. The reinforcement was then put in, the channels were filled, and the floor was put on. In a warehouse upon which he was now engaged—very similar to, but not so large as, those at Shanghai—there was a lift taking up vans and wagons to run on a balcony and discharge the load inside. The balcony was carried on cantilevers, without the pillar used at Shanghai, except in one case, in the lift-well. The columns at Shanghai were very heavily reinforced and were apparently on the Considère system, having spiral reinforcement, but it seemed to him

that the ratio between the cross-sectional area of the vertical bars and that of the spirals was much higher than was usual with the Considère system. He made out that in the columns of the bottom floor there was about 9 per cent. of steel reinforcement, in the next tier of columns about 4½ per cent., and in the upper tier of columns again about 9 per cent. He did not understand why there should be such variation. He believed it would have been better to increase the size of the spiral bars and decrease the size of the verticals. He could not understand from the Figure why the joints of the vertical bars had been made below the floor. He generally endeavoured to get the joints above the floor, so that the concrete could be filled into the columns and beams above them at the same time, right up to the floor-line, leaving the vertical bars standing up for a certain length above the floor. Then the next tier of columns could be placed on that, and so on. He thought there was some danger in not filling the columns and the beams right up to the floor-line. Fig. 13, Plate 2, showed the main beams splayed out from the columns on each side. Sometimes columns were filled up to the bottom of that splay, where the main girder began, and left there. Not long ago he noticed in a warehouse, after the shuttering had been taken down, cracks around several of the columns, just where the beams sprang out. He was rather concerned about it, thinking the cracks were due to stresses set up in the structure; but on examining them closely and cutting them out he found they were filled with a layer of fine coal-dust and sawdust, which in some of the columns was nearly ¼ inch thick. The cause of this was that the columns had been filled up to that point and left, and dust had been blown along the troughs into the depression; it had then been covered by the concrete, forming a serious separation between the two portions of the column. He had the cracks carefully chiselled out and grouted up with good cement mortar. The cost of the superstructure of the warehouse at Shanghai was given as 3*s.* per cubic foot of concrete. Lately he had had some tenders for a similar but smaller building, and in the lowest of the tenders the cost came out to about 2*s.* 7*d.* per cubic foot of concrete. Reinforced-concrete structures were often put out to tender on various systems, and it seemed to him important that the tenders should be made on the same basis if possible: all tenderers should have the same information. Therefore it was very necessary for engineers to design the work themselves as far as possible—not in every detail perhaps, because there were now many firms who had given their whole attention to that particular class of work and

Mr. Read.

Mr. Read. were much more expert in designing details than an engineer would be who had to deal with a reinforced-concrete structure among other work. But certainly the sections of the beams and of the reinforcement should be given. That, of course, was connected with the much-discussed question of the bending-moments, which should be defined for the supports and in the middle. If that procedure were adopted, he thought the tenders would come in on a much more even basis than was often the case now. If the amounts differed by 50 or 100 per cent., there could not be the same quantity of material allowed for in the tenders.

Mr. Hudleston. Mr. F. HUDLESTON remarked that the columns of the new building of The Institution, referred to by Mr. Read, had been designed under the London Building Act of 1909, under which no consideration could be given to the strength of the encasing material. He noticed in all three Papers a good deal of similarity in the stresses taken as working-stresses. Mr. Popplewell had used a design in which the calculated stress in the concrete under compression was 437 lbs. per square inch, and Mr. Ellis had allowed in his plain columns, which were similar to those Mr. Popplewell had used, 450 lbs. per square inch. Both of those figures were about three-quarters of the maximum mentioned in the rules drawn up by a Committee of the Royal Institute of British Architects, which the London County Council proposed to incorporate in their regulations. Mr. Burr suggested that for composite columns the stress might be 500 to 750 lbs. per square inch, and mentioned that his crushing load came out at about 1,500 lbs., or three times to twice the stress he proposed to use. Again, Mr. Ellis in his hooped columns had used a limit of 1,200 lbs. per square inch for the concrete inside the helical winding. The R.I.B.A. rules fixed a limit depending upon the amount of winding put on, up to a possible maximum of 1,600 lbs., with 1 : 2 : 4 concrete and any amount of spiral bracing. That was 33 per cent. higher than Mr. Ellis's working-load ; which seemed to show that at present most engineers designing such work were content with stresses a good deal lower than those which the R.I.B.A. had adopted as the maximum. He thought it was rather a mistake that the London County Council should have adopted somewhat higher figures than those, because when regulations were made everyone worked practically up to the limit at once : he was bound to do so, because he was in competition with others. Personally he thought the R.I.B.A. limit in that matter was a little high. When tensile stress was considered the matter was seen to be somewhat unsatisfactory. The London County Council gave 17,000 lbs. per square inch as the maximum tension to be allowed

in the steel of tie-rods. When an actual stress exceeding 20,000 lbs. per square inch was put upon such rods, the concrete around the bar would probably crack. Taking the test-loads in general use, it was very usual to put on 50 per cent. more than the average live load calculated for, and a very common proportion was one-third dead load, and two-thirds live load. With 3 cwt. per square foot the bars might be strained in tension up to 17,000 lbs. per square inch, but 16,000 lbs. was the limit of the R.I.B.A. If 50 per cent. were added to the live load it would mean a total load of 4, so that the stress on the bars was being increased by one-third, bringing it up to 22,667 lbs. per square inch. He thought that was dangerous, and that if there were limits of stress they should be limits which were not to be exceeded in any test-load put directly on a floor. With regard to the factor of safety, it was distinctly higher in all the cases under discussion. With the plain columns, Mr. Popplewell had a factor of safety of at least 4, his columns being calculated for 13½ tons and showing no sign of breaking at 50 tons. Mr. Ellis's hooped columns were calculated for 1,200 lbs. per square inch, which gave a total of about 78 tons per square foot. In the column Mr. Ellis tested, the working-load by his rules would have amounted to 44 tons and the breaking-load was 139 tons. Mr. Marsh had already referred to the question whether the factor of safety should be referred to the time when the column went to pieces or when it first showed signs of cracking; and Mr. Hudleston agreed that when a column showed signs of cracking it had about reached its limit, and the factor of safety ought to be taken then. That would give in Mr. Ellis's case a factor of safety of about 3. There was another feature of the L.C.C. rules which he thought was a mistake. In them were incorporated a number of rather complex formulas taken almost bodily from the R.I.B.A. rules. On the other hand, Clause 32 said that where the conditions of loading were not specified the bending-moment of the stresses on the slabs or beams must be calculated by the accepted formulas of modern engineering practice. He submitted that it would have been far better if a rule of that description had been inserted in place of all the formulas given. These formulas were not quite the same as those issued 3 years ago by the R.I.B.A. Committee, and it was quite possible that a few years hence they would be altered again. He believed it was a mistake to introduce in such regulations very complex formulas which practically tied the hands of everybody. Again, the preamble spoke of the regulations as referring to buildings in which the stresses were transmitted through each story by a skeleton framework of

Mr. Hudleston.

Mr. Hudleston. reinforced concrete or partly by a skeleton framework of reinforced concrete and partly by a party wall or party walls. It would be a good thing now to have such rules drawn up in a way that would enable steel columns to be combined with reinforced-concrete walls, because it would give a distinct gain in floor-area, which in London was very valuable. Even the most bigoted enthusiast for reinforced columns would admit that a steel column to carry the same load could be made a great deal smaller, that reinforced columns for heavy stresses were rather clumsy, and that they took some time to attain their full strength. If it were possible to combine the two in the new Building Regulations, or to bring in some regulations which would enable the designer to use them without asking for special leave from the London County Council, it would be a great advantage to the profession generally and to all who were concerned in designing buildings. He quite agreed that limits of stress should be given, but he wished to see them rather lower. He suggested that all the formulas which were incorporated in these L.C.C. rules would be better left out, and that a general reference to the R.I.B.A. rules would be much more satisfactory to the profession at large.

Mr. Steinberg. Mr. H. E. Steinberg observed that Figs. 10, Plate 2, of Mr. Ellis's Paper, showed the stirrups to be formed of thin bars twisted together: it seemed to him that this arrangement must have given a great deal of trouble, and he could not conceive any advantage in it. The stirrups in a reinforced beam apparently acted in tension, although they were calculated in shear, and two bars twisted together did not form a very good arrangement for tension, the extension being very great in proportion to the stress. The Paper gave the details of one of the main first-floor beams in the warehouse and showed how a great deal of reinforcement was put over the supports. He thought that if Mr. Ellis had added the eight $2\frac{1}{4}$-inch bars in the columns as well, it would have shown how very complex the armouring at that point was: it must have been very difficult to get the steel into position and to deposit the concrete. With reference to the spiral columns, Mr. Ellis stated that he had adopted the ordinary ratio of 15 between the moduli of elasticity, but he also stated that the spiralled concrete was limited to 1,200 lbs. per square inch inside the core and the steel in compression to 12,000 lbs. per square inch. Surely that was a ratio of 10, not 15. Adopting the new London County Council rules for working out the columns, and taking first of all as the dominant factor the limit of compressive stress in the steel, 12,000 lbs. per square inch, then in the spiralled concrete there would be 800 lbs. per square inch, and the quantity of

armouring provided ($\frac{1}{2}$-inch bars at $2\frac{3}{4}$ inches pitch) would be sufficient to raise the plain concrete from a stress of about 560 lbs. up to 800 lbs. per square inch. It might thus be taken that the basic stress on the plain concrete was 560 lbs., which by the addition of the lateral armouring was enhanced to 800 lbs.; and the longitudinal steel reinforcement, at fifteen times that stress, namely, 12,000 lbs. per square inch, added thereto, would give the total load. But then the total load would be only 303 tons, instead of 376 tons. Looking at it in another way, and adopting the other figure given by Mr. Ellis, namely, 1,200 lbs. per square inch, as the stress on the spiralled concrete, fifteen times that would be 18,000 lbs. per square inch, and the quantity of armouring in the column would indicate that the basic stress on the plain concrete was about 840 lbs. per square inch, which was high. In any case he would like to emphasize what Mr. Read had said, that the columns were not of economical design. It would have been much better, from the point of view of economy and other practical considerations, if the amount of the spiral armouring had been increased and the diameter of the longitudinal armouring had been decreased. By adopting the proportions recommended by Mr. Considère for that type of column, and assuming a reasonable limit for the steel in compression (16,000 lbs. per square inch), and a reasonable basic limit for the plain concrete in compression (600 lbs. per square inch), it would be found that $\frac{3}{4}$-inch spiral bars at about $3\frac{1}{3}$ inches pitch would give about the right amount of spiral reinforcement. The longitudinal armouring might then be reduced from 31·8 square inches, as given by Mr. Ellis, to 21·8 square inches, and the total effect would be quite a considerable reduction in the cost of the column. There was another point he wished to raise with reference to the L.C.C. regulations. As framed they ignored the cover of the concrete outside the spiral. Mr. Ellis stated in his Paper that in the square columns the cover had been counted; but in the spiral columns, for some reason which was not very clear, the cover had been ignored. In a column of such proportions as those between the ground floor and the first floor, by ignoring the cover about 32 per cent. of the gross area of the column was neglected. Subtracting the area of the core from the area of the octagon gave 182 square inches, and taking 450 lbs. per square inch, as adopted by Mr. Ellis, the cover itself was capable of bearing about 37 tons. It would appear to be more reasonable to say, in the L.C.C. regulations, that the area of the concrete inside the spiral might be stressed to a certain figure dependent upon the amount of the spiral armouring, and that the area of the concrete outside the

Mr. Steinberg.

Mr. Steinberg. spiral might be taken into account up to some arbitrary stress, e.g., one-quarter of the crushing strength of the plain concrete. Otherwise, with a small column, say, 10 inches square, it might be better to ignore the lateral armouring altogether, and simply take the 10-inch square and disregard the permissible increased stress on the concrete inside the spiral. With regard to the making of the spiral columns, it seemed to him a very unnecessary procedure to lay the bars laterally round a mandrel and then wind the spiral on them, because directly the spiral was wound it would spring open and the wiring of the longitudinals would have to be done over again. The proper thing to do was to adjust the mandrel to such a size that when the winding was finished the spiral would spring open to the size required. With regard to the wharf, he would like to ask whether it would not have been more reasonable, in view of the fact that the factor of safety of the piles which supported the structure approached unity, to adopt a rather low factor of safety for the superstructure. Why put a structure with a factor of safety of 4 or 5 on the top of piles which were in a foundation that had practically no factor of safety at all?

Mr. Yeatman. Mr. M. E. YEATMAN wished to utter a protest against allowing too great a reduction in the bending-moment for continuity in beams. Mr. Ellis allowed $Wl/12$, where $Wl/8$ would be the moment of an independent beam. One speaker had made out the theoretical figure to be $Wl/13$, but it was by no means safe to take such a large reduction as that. The practice of American and French engineers was to take $Wl/10$. The point was discussed theoretically in Messrs. Turneaure and Maurer's "Principles of Reinforced Concrete Construction"[1] (p. 238), where were shown the moments on two spans and three spans continuous, for various distributions of the load. It had to be remembered that it was necessary to take into account not only a uniform load over all the spans, but the possibility of every alternate span being loaded, or all but one span being loaded, or one span only being loaded; it was necessary also to consider the greatest possible moment, both at the centres of the spans and over the supports, and the reverse moments over the supports were in general greater than the maximum moments in the centre of a span. Therefore, all things considered, the maximum possible theoretical moments were not very much less than $Wl/10$, and that gave a theoretical moment which might be largely upset by any slight unequal settlement of the supports. So that altogether 1/10 was as far as it was safe to go. That was

[1] New York, 1908.

accepted as general practice in America and also, he understood, in France. Mr. Considère, at the International Engineering Congress at St. Louis in 1904, stated that that was the usual French practice. As to Mr. Popplewell's experiments, with all due respect for the care with which they had been made and the accuracy with which the minute compressions had been measured, it seemed to him that they were rather tests of the apparatus than of the behaviour of the steel and the concrete. How could steel and concrete do anything else towards the centre of the span than strain uniformly? If it had been desired to test unequal compression the measurements should have been made at the ends of the column, and then some slip might have been detected. He thought Mr. Popplewell's tests of the grip in concrete were very valuable and interesting. He had been accustomed to require an embedment of fifty times the diameter of a rod before its full strength could be taken up, basing that on an allowance of 75 lbs. per square inch of shearing stress between the steel and the concrete, and 15,000 lbs. per square inch tension on the rod. The experiment pointed to the conclusion that it was useless to allow more than about thirty times the diameter, because the rodding would begin to pull out by contraction of its area, and would pass its yield-point before a greater length than thirty diameters could take effect. The point was that, for a span of a given length, unless anchorage was provided at the ends of the rods, there was a maximum diameter of rod that would give the necessary area. If fifty diameters were required, for a uniform load the diameter of the rods must not be more than $\frac{1}{200}$ of the span, and for a concentrated centre load not more than $\frac{1}{100}$ of the span; but if thirty diameters were allowed it would bring it up to $\frac{1}{120}$ and $\frac{1}{60}$ of the span, which would be a great advantage. Mr. Yeatman.

Mr. Anthony G. Lyster thought it might be interesting to the members to hear what had been done with reinforced concrete in the port of Liverpool, though as he had not expected to take part in the discussion, and had not refreshed his memory in regard to details, he must speak in a general way. His attention was first directed to this form of construction about 13 years ago, when the problem he had to consider was the construction of a cattle-wharf in the Mersey, adjoining the pier-heads. Such a wharf, overhanging the river and needing to be washed down periodically and kept thoroughly clean and sweet, was not easy to provide for with ordinary materials, and therefore he naturally considered the question of the suitability of reinforced concrete for the purpose. He limited the use of reinforced concrete to the Mr. Lyster.

Mr. Lyster. platform, however, as in those days the piles were less trustworthy and much more expensive than they were now. The materials used in the concrete were somewhat rough and coarse, and perhaps at that time the methods of construction left something to be desired, but the work was fairly well done. The steelwork had been exposed to the action of the salt air, and there had been some rusting in patches; but when the defects came to be measured in money the expenditure on repairs was very small indeed, and the wharf had proved entirely satisfactory. The piles were of greenheart. Following that he constructed two other quays which were covered with sheds. Those quays were designed to take a working-load of 3 tons per square yard, and each consisted of a reinforced-concrete platform on greenheart piles. They were tested with a 50-per cent. overload and found to be thoroughly satisfactory, the deflection being within what was prescribed. Another wharf was constructed in which reinforced-concrete piles—the first used in Liverpool—were employed. The bottom was rock, and long spiked shoes were put on the piles. That structure also had been entirely satisfactory. In the later structures, with the experience already gained, greater care had been exercised in the selection of the aggregate, and in the careful disposition of the steel rods and stirrups and bars, and the work, he thought, was of a better character. The next structure was a three-story shed, 628 feet in length and 78 feet in width, on the quay of a dock. The first floor was designed to take a load of 25 cwt., and the second floor one of 16 cwt., per square yard. The roof was flat, the front part of it being reinforced to carry cranes, and the general area strong enough to carry a snow load. One remarkable feature of the structure was the use of cantilevers stretching out over the roadway, which was extremely narrow. An alteration of the dock-quay on the other side of the roadway was contemplated, which would enable the construction of a wider shed in the future, and in order to take full advantage of that he designed the two upper floors to overhang the lower floor by about 16 feet. The cantilevers had to carry all the weight of the upper structure of the two floors as well as their loads. They were tested to 50 per cent. overload, with entirely satisfactory results. After that he constructed another wharf to carry a working-load of 3 tons per square yard, very much on the lines of the others. What he would like to lay stress upon more than anything else, from the point of view of the practical treatment of reinforced concrete, was the extreme accuracy and care with which the work had to be done. The materials, particularly the sand and the broken stone, had to be selected and proportioned with great

care. The disposition of the bars in the girders and columns also required to be very carefully attended to, and steps had to be taken to prevent the bars from being disturbed during the process of concreting. In that respect he thought finality had not yet been reached, and he believed very considerable improvements were likely to be made. His attention had already been drawn to forms of reinforced concrete in which arrangements were made for keeping the bars at their proper distances apart and preventing disturbance of the various constituent parts during the process of construction. He was sorry his memory did not serve him well enough to allow him to give details of costs, but he could say that in the case of the three-story shed he obtained tenders for a steel and brick structure of the ordinary form, as well as for reinforced concrete, and there was a difference of about 20 per cent. in favour of the latter. Alternative tenders recently received for a double-story shed showed a saving of about 30 per cent. in favour of reinforced concrete. A point of great importance had been mentioned by Mr. Yeatman, namely, the reinforcement of continuous beams. That was a matter to which Mr. Lyster's attention had been directed quite recently in considering the designs of a structure on which he was now engaged. No allowance seemed to be made for want of continuity in the system of bars. Obviously where credit was taken for reduction of the bending-moment in a span owing to its being continuous, there must be certainty that the continuity of the system of reinforcement was maintained over the points of support; and that was a very material point, to which those who had to deal with reinforced-concrete structures should devote their attention. Mr. Lyster.

Mr. W. H. Thorpe observed that Mr. Burr's experiments showed an increment of strength of about 45 per cent. in the steel column when concreted. The utility of that method as a means of making the best use of steelwork had already been remarked upon. In a Paper read about 4 years ago [1] on "The New York Rapid-Transit Subway," by Mr. Wm. Barclay Parsons, M. Inst. C.E., experiments were cited which Mr. Parsons had made with rolled steel joists embedded in concrete, and from which he found as a result of the embedment—precisely from what cause he did not attempt to say—an increase of strength, as compared with the normal transverse strength of a steel joist, of about 70 per cent. at 1 month. Quite recently, in the new building-regulations of New York for reinforced-concrete structures—which came into force on the 1st January—it was enacted that where a structural steel column was encased in concrete, though the concrete Mr. Thorpe.

[1] Minutes of Proceedings Inst. C.E., vol. clxxiii, p. 83.

Mr. Thorpe

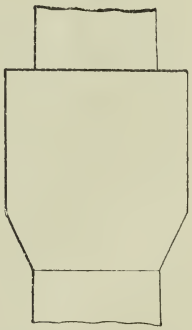
itself was not assigned any compressive resistance, the steelwork in that concrete might be stressed to 16,000 lbs. per square inch, up to a ratio of length to radius of gyration for the steel of 120 : 1. The safe load of such a steel column without concrete would be about 4 tons per square inch, as compared with 7 tons permitted under the new regulations in force in New York. Therefore it was evident that under these regulations the steel was credited with enhanced strength to the extent of about 75 per cent. If that was good practice—and he did not question it—and if the few experimental results which had been obtained were supplemented by others, and it was established that there might be, in the case of steelwork encased in concrete, an enhancement of the strength which might be permitted by the rules of the various authorities controlling such matters to be reckoned upon in design, it gave engineers a means of designing structures which would place steelwork on a more even footing with reinforced concrete properly so called. Reinforced concrete was credited, as the result of a large number of experiments, with its full strength, and he saw no reason why steelwork should not also be credited in the same way with what increment of strength there might be when it was encased, and so placed on a more equitable footing with reference to reinforced concrete.

Mr. Popplewell's results were apparently satisfactory as far as they went, but, as had been pointed out, those displaying the behaviour of the steel with reference to the surrounding concrete at the middle of the column by no means settled the question. Mr. Thorpe had always had serious doubts whether the steel rods in a reinforced-concrete column really took up the stress in the way that was assumed. It might be so in the design tested, where the head and base of the column were enlarged, so that there was a considerable body of concrete around the rods, and where there were spiral hoops to increase the resistance of the concrete and its grip on the steel; but he questioned strongly whether it was so in columns as commonly designed, in which large bars were used and those bars were near the surface of the concrete. Between structural steel and reinforced-concrete work generally there was one marked difference. In ordinary steelwork there were just the same difficulties in the accurate determination of stress as there were with reinforced concrete, assumptions being made which, though convenient, were as little justified in one case as in the other. In a riveted framed structure the joints were treated as if they were hinged joints when they were nothing of the kind. Cross girders were treated as though they were hinged attachments when they were really very different. But no inconvenience commonly

resulted. In most steel-framed structures—he believed Sir Benjamin Baker once said in all steel structures—there was never an entire absence of overstress; some part of the structure was stressed beyond the elastic limit, though as a rule no trouble ensued. There was overstress in particular parts and there was yielding; but the deformation was not progressive; it could not be seen without the use of special instruments, and nothing happened. The structure looked none the worse, and commonly was none the worse. But reinforced concrete when first made was monolithic. There might be movements, reverse moments, stress where stress was supposed not to be, or possibly had been overlooked or considered too trifling in amount to be taken seriously, cracks occurring as a result. There was, he thought, no material, or combination of materials, in which any want of care would so remorselessly "give away" the designer as reinforced concrete. Mr. Thorpe.

Sir William Matthews, K.C.M.G., Past-President, remarked that there were one or two practical points in connection with Mr. Ellis's Paper to which he might refer. First of all he could sympathize with Mr. Ellis in the great difficulties he had had to face in carrying out the work. Those who had had to encounter the difficulties of great depths of silt could appreciate the boldness and the skill with which the work had been designed and carried out. In reading the Paper he had been reminded of a work with which he had something to do at Port Melbourne not long ago, and of what the Victorian engineers had done to increase the bearing-areas of the piles under the conditions found at that place, where a silt bottom only was available, which resembled in character to a remarkable degree the Shanghai bottom. The piles that had been generally used up to then were jarrah piles from Western Australia; they were driven with the small end downwards and chocks were bolted on so that as the pile was driven its resistance to penetration was considerably increased, the bearing-capacity of the pile being, of course, added to materially. It appeared to him that the system of reinforced-concrete construction lent itself to an increase of bearing-area of the piles in a very convenient way, and that by some such method as was shown in *Fig. 1* a considerable increase in the carrying-capacity of the piles might have been provided without any great increase of cost. The other point to Sir William Matthews.

Fig. 1.



Sir William Matthews.

which he wished to refer concerned the platform of the quay-wall. He had occasion about 2 years ago to examine a reinforced-concrete wharf of rather an extensive character, in which there was a platform similar to that at Shanghai. At dead low water he found he could penetrate in a boat right underneath the platform. The filling at the back of the work had sunk away from the deck, and in consequence of the platform the material could not follow it down from the top, with the result that there was a cavity underneath which admitted of a boat going right under the platform. He had risen, however, more particularly to say, as a Member of the Institution Committee on Reinforced Concrete, how very gratified the members of the Committee were to have Papers of this kind brought forward for discussion. He believed it was really the first time The Institution had had Papers on this important subject. There was no doubt that the use of reinforced concrete would spread; in fact, its use was only just beginning. His own firm intended to use it very extensively in the East, because they thought they would get advantages from its use. In carrying out the work they would be making some tests, and it would be their duty, as members of The Institution, to bring the results of those tests before their fellow-members. He quite agreed with Mr. Fitzmaurice as to the value of practical tests made by engineers on works, and he therefore hoped that when any tests were made relating to reinforced concrete—a form of construction as to which some uncertainty existed—they would be brought before The Institution for discussion and inclusion in the Proceedings. The members of the Committee were now engaged in making tests on slabs, and when those were finished they would proceed with tests on continuous beams, which were well worthy of investigation. All those tests required considerable time and entailed heavy expenditure, and he hoped the members would remember that, although the Committee had only issued so far a preliminary report, they were not inactive, but were busily engaged in making tests and had further tests in contemplation. As indicating the extent to which the information they had got together had been appreciated, he might mention that already about 4,300 copies of the report had been issued, and he understood that applications for it were still received pretty regularly.

Mr. Beamish.

Mr. George H. T. Beamish remarked that, from what Sir William Matthews had said, it appeared that before long the members would have a general account of the origin and use of reinforced concrete, and he would like to call attention to an experiment made by Sir Marc Isambard Brunel, at the time of the construction of the Thames Tunnel, with reinforced brickwork.

Brunel succeeded in building a cantilever in brickwork about 60 feet long. It was recorded in the life of that eminent engineer,[1] and as a combination of iron and masonry Mr. Beamish thought it was well worthy of being brought to notice. Mr. Beamish.

The PRESIDENT remarked that he was very glad Mr. Hudleston had made some protest against incorporating very detailed rules in what was practically an Act of Parliament. One instance had been given already, in the course of the discussion, of the way in which such rules, even when they were not very minute, hindered the proper use of the materials by the engineer—namely, the regulations that had been made for steel-framed structures. By those rules engineers were precluded from taking into account the strength of the concrete in which the steel was encased. He did not wish to lay down the law, but it did seem an improper thing that when an engineer was encasing steel in a large quantity of concrete he should be told he must not make any allowance for the strength which the concrete added to the steel. He had been obliged to look a little into the proposed regulations of the London County Council for reinforced concrete, and he thought the practice of incorporating a large number of exceedingly detailed and minute rules reached almost a limit in those regulations. Reinforced concrete was a tolerably new material; although a certain amount of work and experiment had been done with it, and certain conclusions had been reached, it was impossible not to believe that, with further experience, very considerable modifications would be made in the methods of construction which had been adopted so far. When Mr. Considère wound a column with a spiral reinforcement he made almost a revolution in the construction of columns; he showed that the result was a large increase in the stresses which the concrete would carry. It was impossible not to believe that other modifications of that kind would be made. For instance, he had seen lately a modification in the usual method of constructing columns. He had no idea whether it was right or wrong, but it had been used in Scotland on a large scale in one very large building, and it seemed likely to present considerable advantages if it proved to be as good as its inventor supposed. But it would be absolutely shut out from use by the regulations proposed by the London County Council. He wished therefore to reinforce the protest Mr. Hudleston had made against engineers being bound down by too minute regulations. It was easy to see what the driving force had been. Concrete The President.

[1] R. Beamish, "Memoir of the Life of Sir Marc Isambard Brunel," p. 285. London, 1862.

The President. structures, especially buildings, had to be referred to the District Surveyors, men who were used to dealing, and no doubt very competent to deal, with buildings of the ordinary type, but who were less accustomed to deal with the new material; and therefore for their sake very detailed regulations were being laid down, in order that the responsibility might be taken off their shoulders. But it would be a great hindrance to the development of the new material.

Mr. Ellis. Mr. Ellis, in reply, desired to express his thanks to the President and the members for the very kind way in which they had received his Paper. With regard to the small factor of safety in the bearing-capacity of the wharf-piles, Mr. Wentworth-Sheilds had assumed perhaps too hastily that Mr. Ellis's sleep had been undisturbed by the unsatisfactory results of some of the trial loadings of the piles. The risk of subsequent settlement had, of course, been fully recognized, and had been guarded against to a large extent in the design of the superstructure. It must be borne in mind, also, that he had had the advantage of noticing the behaviour under working loads of a number of timber wharves at Shanghai—many of them less adequately supported—and also of watching pile-driving in the early stages of the work in question; thus being able to observe many encouraging indications of the ultimate bearing-power of the river-silt, though it would be difficult to put these on paper in the form of figures. At the present time, after a year's steady use, during which the wharf over the greater part of its area had been frequently stacked with cargo to its full surface capacity, though only locally with its estimated maximum load, no sign of settlement or distress could be observed. The local heavy loading mentioned in the Paper and referred to by Mr. Meik had not been made as a test, but had occurred, owing to some misapprehension of orders, during the ordinary course of working. It consisted of cargo stacked over a deck area of 40 feet by 40 feet, so as to bring a load of about 1,000 lbs. per square foot upon it. As the maximum distributed load had been taken as 350 lbs. per square foot this might be looked upon as a sufficiently severe test; and it was satisfactory to record that no settlement could be detected in consequence. He had been more anxious as to the behaviour of the front groups of piles under the 5-ton travelling crane, when it should be working for some time in one place, and so adding continual vibration to the effect of the maximum load on two piers. Happily, however, no sign of movement was visible there. The moderate unit-costs of the work were due chiefly to the low rates for labour in Shanghai. Credit must also be given, particularly in the case of the wharf, to excellent organization on the part of the contractors, who, it might be

mentioned, were not in any way reinforced-concrete specialists, but a firm of high repute for dock-construction in North Germany. In designing the work, also, simplicity and economy of construction had been constantly kept in view, sometimes at a sacrifice of theoretical economy of materials; and the various structures being on a large scale and of practically uniform design throughout, it had been possible to erect them with the minimum outlay on moulds and false-work generally. Contrary to Mr. Meik's experience, Mr. Ellis was convinced that a decided gain in time, cost, and quality of work was achieved by moulding on shore all wharf-members, except the pile-caps, beams, and deck, which were necessarily monolithic. The costs, on the site, of Portland cement and of steel bars, asked for by Mr. Wentworth-Sheilds, were approximately £2 13*s.* and £9 10*s.* per ton. The somewhat high percentage of steel in the beams, alluded to by Mr. Meik, was due not so much to there being an uneconomical proportion of steel in tension, as to the provision of double reinforcement throughout. The top bars served to resist reverse bending-stresses in a light span between two which were loaded, and also afforded a firm support for the upper ends of the shear members. The heavy reinforcement of the lower warehouse columns was necessary in order to keep their dimensions within the sizes suitable for working purposes. He was inclined to agree with Mr. Vawdrey that $\frac{1}{2}$ inch of concrete cover below the reinforcement of floor-slabs of a building was sufficient. A cover of 1 inch had been fixed for the expanded metal used at Shanghai, because this material was apt to assume a curved form when bent to pass over the supports, and it would thus come dangerously near to the lower surface of the floor at the centre of each span if no margin were allowed. Probably a minimum of $\frac{1}{2}$ inch to $\frac{3}{4}$ inch of cover had been actually obtained in the warehouse floors. In the wharf-deck, on account of its damp situation, 1 inch of cover had been rigorously required. With reference to Mr. Faber's inquiry as to the reason for taking $Wl/12$ as the bending-moment at the centre of the spans of interior beams, that figure had been adopted as a compromise between the theoretical positive bending-moment for beams truly continuous over three spans and that for freely supported beams. It referred directly only to the cross beams, which were designed for a uniformly distributed load, but applied also in principle to the treatment of the girders, the moment-curves being first worked out for continuous beams under the various conditions of loading alluded to by Mr. Faber. The fact that he gave $Wl/13{\cdot}1$ as the maximum bending-moment under the worst possible conditions, while Mr. Yeatman considered it dangerous to take a lower value than $Wl/10$, appeared to show

Mr. Ellis.

Mr. Ellis. that the adoption of $Wl/12$ as a safe mean had some practical justification, whatever might be its correctness in theory. The outside columns were not designed for eccentric loading so far as the arrangement of the reinforcement was concerned. They were rectangular in section, with the longer side in line with the wall, and had a comparatively larger cross-sectional area than the interior columns. With regard to Mr. Smith's suggestion that the wharf might have been further stiffened by a series of longitudinal diagonal members throughout, the provision of these had been considered, but not adopted, partly on the score of expense (nearly £2,000 additional), and also because of the undesirable multiplication of reinforcing rods thereby entailed in the pile-caps and girder-supports where so many members met. In the warehouses it would have been out of the question, from the owners' point of view, to introduce diagonals between the interior columns; and on the outside the existing panel-walls, of 15-inch brickwork in cement mortar, answered the same purpose, so far as a purely local stiffening could be effective. The foundation-grill was expected to be strong enough to withstand any local inequality of settlement, and, so far, had fulfilled this end. He deferred to Mr. Marsh's opinion that the actual maximum stresses in the first-floor columns of the warehouses would be about 1,000 lbs. per square inch on the concrete, and 15,000 lbs. per square inch on the steel. At the same time it appeared not unreasonable to assume that concrete, when confined by close steel hooping, had no longer the same physical characteristics under high pressure as when free to expand laterally, and that m, the ratio of the moduli of elasticity of steel and concrete, might in this case be more nearly 10, as the Author had taken it, than its normal value of 15. In the Table of tests of the wharf-beam, the figure $7\frac{2}{3}$ was not given as the factor of safety, the value of which would approximate to 4, as Mr. Marsh observed, but merely as the relation between the calculated safe superload and the test-load which caused actual destruction of the beam. The comparatively high unit-cost of the buildings, to which Mr. Ball called attention, was due to the expensive foundations, which accounted for nearly one quarter of the total cost. Mr. Etchells was possibly correct in his suggestion that there had been some slight eccentricity of loading in the column-tests. These had been carried out at the works in an ordinary hydraulic testing-machine, and although as much care as possible had been taken over the operation, the test could not be said to have been made under laboratory conditions. It must be remembered, also, that the ends of the tested columns, unlike those in the work, were not fixed in any way. He

agreed with Mr. Read that the piles which had been lengthened and re-driven might well have been left as they were, considering that it had taken eighty blows to re-start them. For this reason only one group had been so treated. With reference to the varying percentages of steel in the warehouse columns, those carrying the first floor had had to be kept of uneconomically small diameter in order not to block working-space: this had entailed a high percentage of reinforcement. The columns carrying the second floor had been allowed to be of almost the same dimensions with a much lighter load, hence making possible a smaller percentage of steel, which was repeated in the columns next above. The joints of the vertical rods had been made below the surface-level of the floors, instead of above them, as Mr. Read preferred, in order to have the lateral support of the concrete floor and beams. This arrangement had not been found to cause any difficulty, the skeletons for the series of columns next above being fixed in place before each floor was concreted; and it gave a stronger joint. Both Mr. Read and Mr. Steinberg considered that the spiral hooping in these lower columns should have been stronger, and the vertical reinforcement correspondingly diminished. He did not follow their argument that sensible economy would have resulted thereby, so long as the diameter of the columns remained the same. The size of the hooping had been fixed at $\frac{1}{2}$ inch for practical reasons, that being the largest diameter of rod which allowed of easy winding round a skeleton of such small sectional area. Mr. Steinberg had apparently misunderstood the description in the Paper of the method of winding this hooping, as none of the drawbacks conjured up by him existed. When the $\frac{1}{2}$-inch rod had been wound round the main bars—a simple and expeditious operation—the skeleton formed a rigid framework ready for erection in place. Similarly, the stirrups of twisted wire to which Mr. Steinberg had objected were adopted on account of their practical advantages, as being readily clinched round the top and bottom reinforcing bars, and forming, with occasional struts, a securely adjusted skeleton. The justification for adopting a factor of safety of 4 for the superstructure of the wharf, while that of the piles was between 1 and 2, was found in the loading incident already referred to, when cargo to the weight of nearly three times the calculated maximum load was stacked over several adjoining bays. The deck had only its own strength to rely on in such periods of exceptional stress, whereas any isolated heavy loading on the piles was distributed over three or more adjacent groups. He much appreciated the sympathetic remarks of Sir William Matthews as to the difficulties

Mr. Ellis.

Mr. Ellis. of the foundations. The hardwood chocks which Sir William had suggested for increasing the bearing-capacity of the piles had been used by him in the case of a steel-pile wharf at Hong-Kong, where they had been found useful for checking excessive penetration into silt and sand. The silt there, however, was of a much stiffer nature (below the surface mud) than the ground at Shanghai, as shown by the fact that screw-piles had been used with success both in Hong-Kong harbour and up the Canton River, whereas at Shanghai they were an entire failure, hardly bearing their own weight. One reason for adhering to the use of plain rectangular piles was that the pile-drivers in use—of a special heavy pattern —were unsuited for driving piles having projections moulded on to them, or chocks bolted to their sides; and in view of the fact that it was apparently skin-friction alone which supported the load, the provision of this additional bearing surface did not appear to offer advantages counterbalancing its drawbacks. The tests showed that when once the mud gripped the concrete there was no further penetration of the piles in a group under any reasonable load. Settlement was caused entirely by the subsidence or compression of the stratum of silt in which the piles were embedded. It was the bearing-capacity of this stratum which determined the maximum load; and this bearing-capacity appeared to be greatest when the silt was compressed by the driving of 14-inch by 14-inch piles in groups. Twelve-inch square piles were experimented with first, but showed comparative inferiority in stability, owing, presumably, not only to smaller area of surface, but also to less severe compression of the silt about them. With reference to the platform of the creek wall to which Sir William Matthews had referred, it was unlikely that much settlement of the ground would take place here, owing to the row of close sheet-piles, the joints of which were grouted solid in front. In any case, the deck was supported by bearing-piles, and was designed to carry the weight of the superincumbent filling, so that no disastrous results should follow the formation of cavities beneath it.

Mr. Popplewell. Mr. Popplewell, in reply, thanked the President for his remarks, and expressed his gratification at the way in which his Paper had been received and discussed. Referring to Mr. Meik's remarks, he had used the word "adhesion" in his Paper as suggesting some property peculiar to the concrete, enabling it to attach itself to the steel in a manner similar to that in which two pieces of wood were held together by glue. He had intended to indicate that in his opinion there was no such property, and that the resistance to the slipping of the bars was due to purely mechanical friction caused by

the fluid material setting against the rough surface of the steel and afterwards being forcibly held against the bar by shrinkage. The variation in the values obtained for the resistance to slip per unit of area might have been due partly to a difficulty in determining the length of the sliding portion of the bar beyond that which had been thinned by yielding. In this connection the figures in the following Table might be interesting as showing the frictional resistance of bars which, instead of being pulled out of the concrete, were pushed through it. The concrete in this case had the same composition as that described in the Paper. Mr. Popplewell.

Results of Gripping Tests, in which Specimens of Round Steel were Pushed through Prisms of Concrete 6 Inches Square in Section and of Various Lengths.

Size of Bar.	Length of Prism.	Age of Concrete.	Frictional Resistance in Lbs. per Square Inch of Surface.	Average.
Inch.	Inches.	Months.		
$\frac{9}{16}$	6	4	422	
,,	7	,,	471	
,,	8	,,	675	
,,	9	,,	481	587
,,	10	,,	590	
,,	11	,,	790	
,,	12	,,	660	
$\frac{3}{4}$	6	9	660	
,,	7	,,	642	
,,	8	,,	680	
,,	9	,,	520	635
,,	10	,,	505	
,,	11	,,	695	
,,	12	,,	743	

He failed to appreciate Mr. Meik's remark with regard to column C; it was stated in the Paper that column A failed at 44·5 tons, while columns B, D, and E, were loaded up to 50 tons, at which load they were probably not far from failure. It was not possible to account definitely for the difference in the result obtained in the crushing-tests of the cubes. It was well known that these differences did occur when a number of cubes were tested which had the same composition but which had been made at different times. Unless

Mr. Popplewell.

the conditions were made uniform in every respect, uniformity of results could not be guaranteed. As an example of this the following figures might be quoted as showing how specimens might vary in strength, although of the same composition and age, when the length of time between the mixing and the moulding was varied. These tests had been carried out by Mr. Popplewell for Mr. Corbett, of Manchester[1]:—

Age of Test-Specimen.	Time between Mixing and Moulding.			
	Fresh.	2 Hours.	4 Hours.	6 Hours.
2 months	192	174	150	126
4 months	234	183	172	167

Mr. Marsh's difficulty with regard to the gauge-length was apparently due to the fact that the actual measurements taken with the Martens extensometer were on 20 centimetres or 7·98 inches, and for convenience of reference and calculation the compressions given on the diagrams were their equivalents on lengths of 12 inches. Mr. Etchells had expressed surprise at the difference between the limit-of-proportionality stress and the yield-point stress of the steel used, but in the light of his experience of many samples of mild steel tested Mr. Popplewell did not consider this result in any way remarkable. The yield-point mentioned on p. 36 as 7·5 to 7·96 tons referred to the total load, while the yield-point of 18 tons, given on a previous page when describing the properties of the steel, would be seen on examination to refer to the load in tons per square inch. He heartily endorsed Mr. Faber's opinion as to the form which should be given to the ends of the test-columns in reinforced concrete, and agreed that the part of the column where the frictional grip was such as to transmit only a part of full load to the steel—so throwing on the concrete more than its share of the load—should be enlarged in cross section so as to reduce the stress in the concrete to something nearer its normal value. He could not admit the validity of the contention put forward by Messrs. Vawdrey and Yeatman that the concrete and steel in the middle of a column such as that described in the Paper could not do otherwise than compress together as one material, or, in other words, that the steel could not fail to carry its legitimate stress as fixed by the ratio of the two moduli. If there were no frictional bond between

[1] Journal of the Royal Institute of British Architects, 3rd series, vol. xviii, p. 18.

the steel and concrete, or if this bond were very weak, then evidently the pressure of the concrete on the ends of the rods would not be sufficient to develop the full amount of stress in the steel. Either the frictional grip of the concrete on the steel must be sufficient to carry the full load of the bars at their centre, or, if this friction was not sufficient, the ends of the bars must abut against an unyielding surface, compelling equal strain of the concrete and the steel. If these conditions did not obtain, then it was conceivable that, as the load came on the column and the concrete was compressed, it would be pushed past the ends of the rods, which would compress the concrete immediately in contact with them. From the present knowledge of the properties of reinforced concrete most people would expect the concrete and steel to behave as they were shown to do in the experiments; but to say that they could not possibly do anything else was surely going too far. Mr. Yeatman's suggestion that measurements might have been taken nearer the ends was a good one, and it might be well in future experiments of this kind to go a step farther even than this and take measurements at, say, three places on the bar. This would furnish information as to how the frictional grip affected the stress in the bars throughout their length. The stress would be expected to be greatest in the centre of the column, tapering off to something very much less at the ends. Several speakers, notably Mr. Faber and Mr. Marsh, had remarked on the lowness of the modulus of the concrete. He had determined the modulus in question by means of Martens mirror observations taken on a prism of plain concrete having the same composition as that used in the columns. This particular concrete was not taken from the same mixing as any one of the columns, but was made of the same materials and as nearly as possible under the same conditions. At the same time it was quite possible that there was some variation in the conditions under which it was made that tended to lower the value of its modulus. He repeated his determination on the same prism about a year later than the second experiment mentioned in the Paper, that was to say, when it was about 18 months old: he found that the material had hardened still more, and obtained a modulus of 1,900,000 lbs. per square inch. With the modulus found for the steel, this would give a value of nearly 16 for the ratio of the moduli, m, which was much nearer the 15 mentioned by Mr. Faber and was the modulus found by Mr. Marsh's calculations. The modulus for steel had a value which varied only within comparatively small limits for all steels, so that the variation in the ratio depended almost entirely on the modulus of the concrete.

Mr. Popplewell.

Mr. Popplewell. This was known to vary, even for concretes which were supposed to be the same in composition and age, within very wide limits, and he thought it unfair to base all calculations on an assumed constant value of m. In the absence of direct experiment at the time, it was impossible for designers to avoid making some such assumption, and, as Mr. Wentworth-Sheilds pointed out, even a large difference in the value of m made relatively small differences in the loads which columns could safely bear. In his opinion the values generally assumed for m had a tendency to err in the direction of being too low. In conclusion, he thought that further experiments might be made on similar lines to those described, especially in the direction of taking linear measurements at different parts of the bars, comparing the effects of central loads, eccentric loads, and loads uniformly distributed on the ends, and noting the effect of loadings repeated many times. It would also be of benefit to those carrying out reinforced-concrete work if the methods of carrying out crushing-tests on concrete could be standardized to some extent.

Mr. Burr. Mr. Burr, in reply, remarked that, in view of some features of this highly interesting discussion, it seemed advisable to direct attention to the fact that his tests had not been intended to accomplish more than to secure information at one point in what might be called a particular problem in reinforced-concrete column-work. A number of circumstances rendered it advisable to use the relative sectional areas of concrete and steel found in the columns tested. It was realized that the percentage of steel section was high, although not as high as had been used in actual work where this type of column was employed for high buildings. Indeed, the percentage of reinforcement in some columns of the usual banded type described in the discussion was so nearly equal to that found in these columns that the difference was immaterial so far as the ratio between the steel and concrete sections was concerned. The percentages of concrete and steel in the sections employed were not in any sense indications of what he might consider most advisable for any particular case. He hoped eventually to extend this experimental investigation into a series in which the percentage of steel would range from the lowest values employed to perhaps higher than in these tests. Mr. Meik and some others had objected to the term "reinforced concrete" being applied to columns of the composite type; but it was a matter of little consequence what this type of column might be called. It was probably reasonable to consider any combination member or structure composed of two different materials a composite member, including what were universally known as reinforced-concrete beams. Abundant experience with

conflagrations in tall steel-frame buildings in New York City had shown the effectiveness of a shell of such material as concrete outside of metal columns as fire-resisting protection to the covered metal, although the impact of heavy fire-streams and the falling of debris would occasionally crack it and detach it in places. This partial destruction in a protracted fire would render it improper to consider such a shell as part of the supporting material of the column, although effective as the fire-protection for which it was designed. It was for this reason that the compressive carrying-capacity of the concrete within the exterior limits only of the steel framework of the column was recognized. There was a large area of continuous connection between the mass of concrete in the interior of the column and the exterior shell, sufficient to give the latter all the support required for protective purposes. This might be supplemented, if desired, by wire wound rather loosely about the steel reinforcement, which would act as a kind of wire lathing to hold the exterior concrete or mortar protecting shell in place during the destructive vicissitudes of a protracted fire. Mr. Vawdrey would probably not consider the round rods and spiral or other winding wire of the usual wire-wound reinforced column as capable of carrying any material amount of compression as a steel column in itself, and that was what was meant by the quotation which he made from the bottom of the first page of Mr. Burr's Paper. Such a cage of steelwork, acting independently of the concrete in which it was encased, would certainly be markedly ineffective as "a direct load-carrying element of a column." The point Mr. Burr desired to make was that for high buildings—perhaps five or six to thirty or forty stories in height—it was essential to use a column capable of sustaining the bending stresses developed in the use of long columns. It would probably be generally admitted that this could be satisfactorily accomplished only by the use of a form of steel reinforcement capable of meeting such requirements by its inherent qualities as an independent member. The presence of concrete would give enhanced resistance to direct compression, but certainly not to long-column bending, except possibly indirectly to a limited extent. In fact, Mr. Vawdrey essentially concurred in this view in his remarks. In this connection it was well to point out that the straight straps or lacing bars between embracing members of steel reinforcement of the type of column under consideration formed but one part, and frequently not the main part, of the lateral support given by the steel to the enclosed concrete. The corner steel angles or the steel channels, or other similar parts used in this type of column, were frequently and perhaps generally the main lateral supports of the concrete,

Mr. Burr.

Mr. Burr. continuously and materially embracing the latter. The point was raised in Mr. Faber's interesting observations that the mode of application of the load at the end of a reinforced-concrete column would affect the relative carrying-capacity of the steel and concrete; and that view was undoubtedly correct. It was Mr. Burr's opinion that the type of column covered by his tests possessed a marked advantage in being adapted to such detailing at floor-intersections and other points of application of loading as to convey more effectively the designed proportions of load to the steel and concrete than was the case with spirally-wound or other similarly reinforced members. Regarding the unit working-stress to be allowed on the concrete of reinforced-concrete columns, to which Mr. Marsh made a very apt and suitable reference, it must obviously depend upon the amount of lateral support given by the banding or encasing steel employed. If there was a small amount of banding, spiral or other, or if the load-carrying steel of the other type of column was very light, the working-stress should necessarily be smaller than would be justifiable with a much higher percentage of the reinforcing steel, either as banding or as an encasing column. Spiral or plain banding had been employed by Mr. Considère up to 12·9 per cent. of lateral and 1·2 per cent. of direct reinforcement, giving to the concrete an ultimate resistance as high as 25,600 lbs. per square inch; but that could scarcely be considered a practicable reinforced-concrete column for ordinary building purposes. Even sand might be so encased or held by a thin cylinder of steel, or even by close spiral winding, as to carry a relatively high load per square inch. He concurred with Mr. Marsh in the opinion that the concrete in reinforced-concrete columns ought to be so supported as to yield a higher working-stress, especially in large structures, than 500 to 700 lbs. per square inch. He believed that in some structures it should be possible to reach a working-load of 1,000 lbs. per square inch on the concrete of reinforced-concrete compression members. The observations of Mr. Etchells regarding the classification of reinforced-concrete columns appeared to be well grounded. As had already been stated it seemed to be of little consequence whether this particular type of column were called a composite column or a column of reinforced concrete, since, as already observed, any combination of two materials in one member made that member composite. He believed from his own experience of the use of this type of column in high-building construction that the care required to secure the satisfactory filling of the interior spaces was no more than was compatible with economy and rapidity of construction. A relatively wet concrete and thorough agitation would accomplish the desired end, the largest individuals of the

aggregate being small enough to cause no difficulty in securing a dense concrete core. In commenting upon the failure of the two types of steel columns Mr. Etchells appeared to have fallen into an error in the computations relating to the steel-channel columns. He gave the ratio of the length of the steel channel between wrappings, divided by the least radius of gyration of the channel section, as 50. As a matter of fact, the 4-inch by $\frac{1}{4}$-inch wrappings were 16 inches apart between centres, or 14 inches between the centres of the fastenings which determined the length of column. As the least radius of gyration of the 3-inch channel was 0·41 inch, the ratio l/r was 34, and not 50; hence Mr. Etchells's observation regarding the failure of this column in detail was not sustained. Further, as stated in the Paper, these channel-steel columns failed by general flexure rather than by detail flexure of any part of the channels. Mr. Wentworth-Sheilds had touched upon an important point in raising the question of a proper ratio between the moduli of elasticity for steel and concrete. Undoubtedly there were many well-established experimental results which indicated that this ratio might be taken as low as 12 or even less, while, on the other hand, other reliable experimental results might justify taking the ratio as high as 20, even for 1 : 2 : 4 concrete with cement of excellent quality. Obviously the value of the ratio would depend upon the age of the concrete, other things being equal. It would appear reasonable, however, in view of the large number of determinations which had been made so far, and in view of the further fact that reinforced-concrete members or structures must frequently sustain considerable loading at an age not greater than 30 days, to conclude that the prevailing value of 15 was well considered and worthy of its wide use. Mr. Burr was pleased to notice that Mr. Fitzmaurice concurred in his view as to the classification of the two types of columns considered in his Paper. It was difficult to imagine any substantial reason for not calling both types reinforced concrete. The same materials were elements of both types, and there was not the slightest doubt as to the mode of combination or as to the significance of any reference to either. Mr. Fitzmaurice's familiarity with high-building construction in New York City had prompted him to remark very aptly upon the adaptability of the two types of columns. The space occupied by columns in the lower floors of buildings ten to thirty or forty stories high was a matter of the utmost importance where floor-space was so valuable as it was in New York City. This fact, together with the capacity of the so-called composite type of column to resist column flexure, gave it adaptability to buildings of any height; whereas the larger area of

Mr. Burr.

Mr. Burr. cross section required for the spiral-wound or other banded column of the usual type and its lack of capacity to resist the bending action arising in long columns practically ruled it out of use in high-building construction, especially for buildings intended to contain heavy running machinery such as printing-presses, which set up vibrations that were frequently severe. In answer to Mr. Read's question how much of the extra strength of concrete-filled columns was relied upon in the calculations for the columns in the high buildings in New York, it was customary to use the full working-load permitted on concrete. In one eleven-story building in New York City where the columns and other structural work had been designed and constructed under Mr. Burr's direction, a working-load of 750 lbs. per square inch had been adopted for the concrete part of the composite columns, that load being permitted by the regulations of the Bureau of Buildings of New York City. If no additional load-carrying capacity could be credited to the concrete part of the section, few or no American engineers would use the concrete, but would design the regular or standard steel columns without any filling whatever.

Correspondence.

Mr. Bergin. Mr. W. M. Bergin considered that Mr. Ellis was to be congratulated on having carried out a work of such magnitude in reinforced concrete in China, but regretted that on some points a little more detail had not been given. Nothing was said as to how the piles were handled; as the largest must have weighed 7 or 8 tons it must have been no small matter to move them about and place them in position ready for driving. In the design of the wharf-deck there was what seemed to be a serious fault, namely, the absence of any provision for expansion and contraction. Apparently an attempt had been made to construct a reinforced-concrete monolithic slab measuring 1,160 feet by 174 feet and subject to a range of temperature of about 150° F. (Shanghai winter-night temperature to summer-sun temperature), with the inevitable result that cracks occurred at or near the points where expansion-joints for the concrete should have been made, and exposure to corrosion and ultimate destruction of the reinforcement at these places was only a matter of time. The design of the reinforcement—a succession of connected cantilevers—excluded the

possibility of expansion-joints, and for that reason should be condemned for such work. Separate units, each self-contained and capable of expanding and contracting without cracking, would have ensured a permanent covering and protection for the metal. From experiments on 1 : 2 : 4 concrete, the coefficient of expansion, as given by the best authorities, was 0·0000055 per degree Fahrenheit; this would give 11½ inches for the wharf-deck exposed to the direct rays of the sun, but only about half as much for the supporting members. No provision had been, or could have been, made for this in the design used, but it could have been readily provided for with rolled-joist instead of bar reinforcement, divided into bays and panels. Each bay could be completed in one making, thus securing joints where intended and at those places only. Mr. Ellis's conclusions as to the excellence of concrete made by Chinese coolies trained to the work would be immediately endorsed by all who had had to deal with them; even when all mixing had to be done by hand, the result was certainly equal, if not superior, to that done under the most advantageous conditions by English labourers. Though there was no single work of magnitude to compare with that described in Mr. Ellis's Paper, a considerable amount of interesting reinforced-concrete work had been carried out on the Chinese Government Railways (formerly the Imperial Railways of North China). The first example was carried out in 1903, for the railway gateways in the walls of Peking. Subsequently this material was used for subway-coverings and engine-shed roofs, and concrete piles for curtain-walls were driven to protect small culverts from scour. Recently reinforced-concrete bridges of several spans, for flood-openings, had been introduced, but hitherto they had been built only where the embankment was not high nor the scour deep. They were of extremely simple construction, consisting of reinforced-concrete piles with reinforced-concrete caps to form the piers, which carried a reinforced decking continuous over two 10-foot spans. The following were the principal details, which might be of general interest. The piles were of 15-inch square section, composed of 1 : 2 : 4 concrete, and reinforced with four 1-inch square rods braced together with ¼-inch round iron. The piles had a central pipe for use with a water-jet, terminating at the point in a small iron shoe with a ¾-inch hole. These piles were generally allowed about 3 months to set before driving, but were, in some instances, driven when only 6 weeks old, with no bad results. The cement used was a Portland cement manufactured in South China, which proved quite satisfactory; the sand was a coarse, sharp river-sand, and the ballast consisted of ¾-inch hard limestone chips. The lengths of

Mr. Bergin.

Mr. Bergin the piles varied from 16 feet upwards, depending on the height of the embankment, the kind of ground, and the scour expected. A steam piling-plant not being available, the piles were driven with an ordinary drop-monkey piling-engine, worked by a steam-winch on the engine-frame. The monkey weighed 2 tons, and driving was assisted by the water-jet, water being supplied by a small steam-pump giving a pressure of about 75 lbs. per square inch at the nozzle. At the start, when the monkey was merely resting on the pile-head, with the jet full on, the pile sank about 12 inches, and the first few blows (6 inches drop) drove 3 to 4 inches each. At about the tenth blow a 9-inch drop drove 2 inches. The total distance actually driven was about 10 feet, and when driving was nearly completed a 24-inch drop drove 1 inch with the jet full on. The soil was a stiff black mud overlying hard yellow clay. The driving-cap consisted of a square ring of 6-inch by $\frac{1}{2}$-inch iron, loosely fitting the pile and supported on the pile-head by a small internal angle. A flat piece of wood 2 inches thick rested directly on the pile-head inside and clear of the angle-bar; on this was a $\frac{1}{4}$-inch iron plate fitting the ring loosely, and on this again a second piece of board 2 inches thick, projecting 1 inch above the top of the ring. Four iron legs riveted on to the outside of the ring and projecting downwards gripped the sides of the pile and held the ring steady in place. The blows were delivered directly on the top piece of board, which had to be renewed usually for every three or four piles, while the lower piece lasted for about a dozen piles. In no case was the slightest damage done to the heads of the piles. The rate of actual driving was five or ten times quicker with the water-jet than in the earlier bridges where no jet was used. The use of a water-jet was therefore to be strongly recommended whenever possible with reinforced piles, as these lent themselves so readily in their construction to the introduction of a central pipe; the chances of injury while driving were also eliminated, and no complicated power-absorbing driving-caps were necessary for hard ground. In very soft ground, such as at Shanghai, piles could probably be sunk by jet alone, or with the assistance of a very light hammer. The piers were of three kinds. The main piers consisted of a row of four piles 3 feet 3 inches apart from centre to centre, with a reinforced-concrete cap 12 feet 6 inches long by 6 feet deep and 21 inches wide, carrying the ends of the deck units (*Fig.* 2). The intermediate piers had similar piles, but the cap was only 3 feet deep and 18 inches wide; the abutment-piers contained six piles in a row with a cap similar to the main piers but proportionately longer. The concrete in the pile-heads was cut away to expose the reinforcement, and cross

reinforcement, consisting of two 15-lb. scrap rails, was wired in place, the caps being made in situ. The decking consisted of flat-bottomed concrete troughs continuous over two spans and reinforced with four 14-inch by 6-inch steel joists, spaced 2 feet 10 inches apart. Two 60-lb. scrap rails were placed in each ballast-wall, the whole being made in situ after the caps had set (*Fig. 3*). The concrete for piers, caps and deck was mixed in the proportion of 1 : 3 : 6, the ballast ranging from $\frac{3}{4}$ inch to $2\frac{1}{2}$ inches in size. The deck might appear strong, but for small spans, with the shaking caused by the passage of heavy locomotives at high speeds, the reinforcement could not be cut down to a few rods. The cost worked out at about $12\frac{1}{2}$ per cent. less than the former standard type—with timber piles and concrete piers to carry plate-girders—but its great superiority lay in the fact that it gave a continuous ballasted track, and the maintenance was practically nothing. Quite recently, reinforced-concrete signal-posts had been introduced and were now gradually replacing the old timber posts. They were 31 feet $6\frac{3}{4}$ inches long over all, and

Mr. Bergin.

Fig. 2.

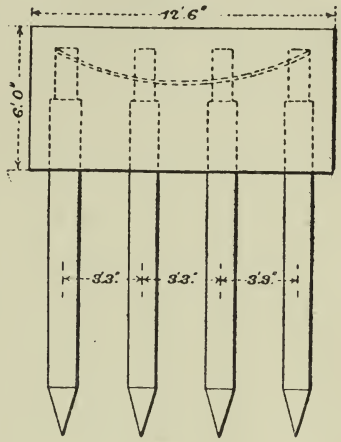
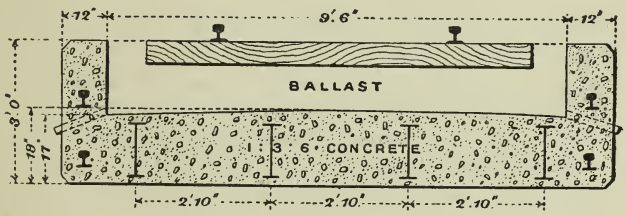


Fig. 3.



$10\frac{3}{4}$ inches square at the base, tapering to 7 inches square at the top. The reinforcement consisted of eight $\frac{1}{2}$-inch square bars for the lower 22 feet, the remainder having only four similar bars, all being braced together with $\frac{3}{16}$-inch iron wire. Holes were left, clear of the reinforcement, for bolting on all old standard fittings. Lamp-posts with the simple and economical reinforcement of one scrap boiler-tube

Mr. Bergin. up the centre were in use, and were found eminently satisfactory. The latest reinforced-concrete structure to be introduced was a "whistle" post. This consisted of an upright post 10 feet high over all, reinforced with four $\frac{3}{8}$-inch round bars, and carrying, flush with one face, a vertical slab reinforced with expanded metal. The whole was moulded in one piece with raised Chinese characters notifying to the driver to whistle. All the foregoing works had been designed by Mr. L. J. Newmarch, M. Inst. C.E., the Senior District Engineer at Shanhaikuan, and they had given great satisfaction in use.

Mr. Brenan. Mr. G. Woulfe Brenan remarked that the piling adopted for the Shanghai wharves seemed to have been successful, in spite of the fact that the surface bed lay upon a depth of 300 feet of mud. The assumed maximum bearing-capacity of each pile was given on p. 5 as 5 cwt. per superficial foot of the pile. A 14-inch square pile driven 25 feet into the river-bed had therefore an estimated bearing-capacity of about 30 tons, and in a bad place four piles came to rest after a month's test-load of 45 tons, or $11\frac{1}{4}$ tons per pile, giving a factor of safety of about 3. In constructing 2 years ago the seaward end of a slipway which was encased within reinforced-concrete sheet-piling, 14 inches by 6 inches in section, Mr. Brenan had to deal with a soft clay bed of unknown depth. A steel rod 1 inch in diameter found no firm bottom at 20 feet of depth. This water-borne clay was covered on top with a layer of small stones, gravel, and clay, 2 feet thick. The sheet-piles went through this bed at the rate of 1 inch per blow from a 2-ton hammer, but below that depth they sunk with the weight of the hammer and dolly resting upon them. There were four 12-inch temporary pitch-pine piles driven in the same place, which carried the floor for the winch and pile-hammer, and noticing that these had firm hold after some weeks in position, he had the sheet-piles fixed by lashings for 24 hours. They were then found to be securely held by friction, which enabled the pile casing to proceed to completion without any further sinking. The weight held by skin-friction in this case, after 24 hours, was 1·4 ton, or 0·625 cwt. per superficial foot as the pile sank in the ground. The particulars given in the Paper were worthy of note as an example of a kind of sea-bed not at all uncommon, even on a rock-bound coast, where water-borne clay took the place of mud. In the course of building some reinforced-concrete piers for the use of coasting-steamers having limited time for stoppage, he had formed the opinion that the concrete pile-fronts required considerable protection. This was effected by means of timber fender-piles, walings, and coverings, to save injury

to the concrete beams and members, should they be struck by steamers coming to the pier. The illustrations of the work at Shanghai did not show any details of such timber fenderings; nevertheless they were of importance, and would play no small part in maintaining the security of the concrete structure. The reinforced-concrete piles or columns themselves would not stand a blow from a steamer without injury. Several cases had occurred recently in which complete horizontal fracture of the concrete part of 14-inch piles had taken place from this cause. Further, 12-inch fender-piles projecting beyond the face of the concrete piles could not be secured by bolts alone: they must be held in position also by heavy timber walings, or they would be twisted out of place, and the bolts either sheared or bent. This had occurred to his knowledge on several occasions. The necessity for thus fendering the working-fronts of concrete piers did not seem to have had sufficient consideration up to the present time, and it could not be doubted that the cost of this fendering, and of its renewal from time to time, would weigh somewhat against the advantages offered by the use of reinforced-concrete piles. The essential weakness of a 14-inch concrete pile appeared to be due to the fact that, relying on the strength of the flooring, the piles were spaced too far apart, say, at 12 to 15 feet. Further a 14-inch section of pile or column was too small to encase properly the steel at the central junctions, which required as many as forty bars within the 14-inch square. Thus in many cases there was not $\frac{1}{2}$ inch of concrete covering, and, still more important, there was not $\frac{1}{2}$ inch of space between some pairs of the bars themselves. It was practically impossible to fill such a combination with concrete. If it were attempted, voids would be caused by small stones jamming between the bars. The only thing to be done was to grout up the part with 1 : 1 stiff cement grout, or mastic, as it ought to be called. It should be remembered that in sea-work these parts had to be made up in all possible exigencies, between wind and water, and within very limited time; so that reliable homogeneous construction was at most a doubtful possibility. In the works described by Mr. Ellis the columns had been brought up with sections larger than the piles, which had probably been done to meet the foregoing objections. There were other points worthy of notice in connection with reinforced-concrete work on piers. The steel rods in the decking should never be less than 1 inch below the floor-surface, otherwise when the men trod on the ends of the rods, during the concreting, the bars would spring upwards and burst the concrete. All stirrups should be pulled up tightly to their

Mr. Brenan.

Mr. Brenan. respective bars after the concrete had been rammed in to a certain height, otherwise the bottom of the stirrup would be pushed down below its bar in the operation of packing, thus rendering it useless for its purpose, and probably causing it to show itself on the underside of the beam, to rust away in a short time. In the building-up of columns all links should be wired to the vertical bars, not merely laid in position, as they were liable to be rammed down one upon another, so that instead of being 6 inches apart they would be found to be 2 feet in some cases. If the under-water beams and other members were made ashore, specially strong concrete should be insisted upon in making the junction, first well painting the meeting parts with strong grout. It was of decided advantage to a contractor to build the beams ashore. It saved much time and risk, but clearly it did not produce as strong a beam as building all up simultaneously in situ. A jointed beam depended for continuity on the steel alone, the concrete being merely a protection against the deformation of the steel. It was a matter of the greatest difficulty to detect all the errors that could be made among a gang of even trained labourers in carrying out reinforced-concrete works. The only remedy was constant and individual inspection of every beam, column, or member, during construction and fixing.

Mr. Buel. Mr. Albert W. Buel, of New York, thought that the tests described by Mr. Burr were interesting as giving results not very different from what might have been expected. They were, however, somewhat disappointing, in that they did not seem to have developed the possibilities of the materials. In the light of tests previously made on columns of wrought iron, structural steel, plain concrete sand reinforced concrete, the designs of these test-columns hardly seemed to have attained such efficiency or economy as might reasonably be expected. He was in entire agreement with the preference expressed by Mr. Burr for a type of steel reinforcement which was a direct load-carrying member in itself, but would add—at least for important structures or high buildings—the requirement that the steel should be so designed and proportioned as to be able to carry, without assistance or support from the concrete and without exceeding the elastic limit of the metal, all the loads that might come upon the column. The concrete filling, properly hooped or banded, would then add greatly to the strength and safe working-load of the member. This would permit the structural-steel column, embedded in concrete, to be designed considerably lighter than would be required if the strength of the concrete were neglected. At the same time, the steel element alone would have

sufficient strength to carry all the loads if, from any cause, the concrete should fail. A comparison of Mr. Burr's tests with a number that had been made on plain concrete columns of similar mixture (1 : 2 : 4), did not seem to indicate that the load-carrying capacity of the concrete itself had been materially increased by the steel reinforcement. Tests made at Watertown Arsenal [1] had shown a compressive strength for concrete of 1,413, 1,710, 1,750, 1,990, and 1,506 lbs. per square inch. More recent tests made at the University of Wisconsin [2] had shown an average strength of 4,459 lbs. per square inch for cylindrical columns reinforced with four vertical rods and helical winding; 3,879 lbs. per square inch for square columns, also reinforced with four rods and helical winding; and 2,880 lbs. per square inch for square columns reinforced with four bars banded. Definite conclusions could not be drawn from these later tests until more complete data were at hand, but on the face of the published reports it appeared that the square columns with helical winding were about 35 per cent. stronger than the square columns with the vertical bars banded, and that the cylindrical columns with helical windings were about 55 per cent. stronger. The concrete used in these tests had the same ratio of cement to sand and stone as that used by Mr. Burr. The length of the columns was $7\frac{1}{2}$ feet. The cylindrical columns were $8\frac{1}{4}$ inches in diameter, and the square columns measured 10 inches by 10 inches. As the value of hooping or banding lay in its resistance to the lateral deformation of the concrete, or in its restraint of the tendency of the sectional area to increase under compression, it seemed almost self-evident that the bands or hoops should always have a circular cross section, or take the form of helical windings on a cylinder. Any other form would fail to restrain the concrete efficiently, as the bands or hoops would bend or bulge before their tensile resistance came into effective action. Mr. Burr did not give the ultimate stresses in the concrete, but taking E_c at the last values noted, which were doubtless too high, they never exceeded 1,950 lbs. per square inch, while properly hooped columns of 1 : 2 : 4 concrete generally carried ultimate loads of 4,000 to 5,000 lbs. per square inch. The conclusion that the banding was not very effective seemed clear. It was important to note, however, that the pitch or spacing of true hooping should not much exceed one-sixth of the diameter. Mr. Burr's statement, as applied to the design of his test-columns, that the steel reinforcement effectively enwrapped or banded the

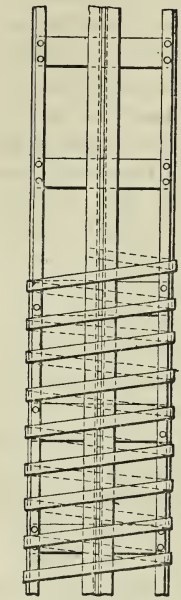
Mr. Buel.

[1] "Tests of Metals at Watertown Arsenal." Washington, 1904–6.

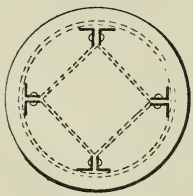
[2] *Engineering Record*, vol. 65, (1912), p. 87.

Mr. Buel. concrete so as to give it the greatest possible added carrying-capacity which any kind of banding could produce, did not seem to be at all justified by the tests reported. Mr. Buel saw no great difficulty in the problem of combining in one design all the advantages of a steel reinforcement which, independent of the concrete, would be a direct load-carrying member, with those of a thoroughly and properly hooped column. He suggested the design shown in *Fig. 4* as one way in which it might be solved. The longitudinal reinforcement, consisting of four or eight angle-bars connected with tie-plates or battens, was similar to the steel column designed by Mr. J. H. Gray, which had been used in many important buildings in the United States. It was very economical in material, could be constructed at a low cost, and facilitated good design of details and connections. It was particularly suited for use with helical winding or hooping, as he had endeavoured to indicate in the Figure. The concrete inside the hooping might be mixed as lean as 1 : 3 : 6, thus effecting a considerable economy in cement, without much detriment to the strength of the column, as the ultimate strength of the longitudinal steel elements would probably be reached before that of the hooped concrete. This was so even with the leaner mixture, as the effect of true hooping was relatively more efficient with such concrete. If the same principle were applied to the design of girders and beams—making the steel reinforcement in the form of direct load-carrying members independent of the concrete—the entire steel frame could be erected in advance of the concrete work. It would not only be self-supporting, but might easily be designed to carry all the centering and forms, as well as the necessary construction-plant. The design of column suggested lent itself

Figs. 4.



ELEVATION OF REINFORCING METAL



SECTION OF COLUMN

Scale 1 Inch = 2 Feet

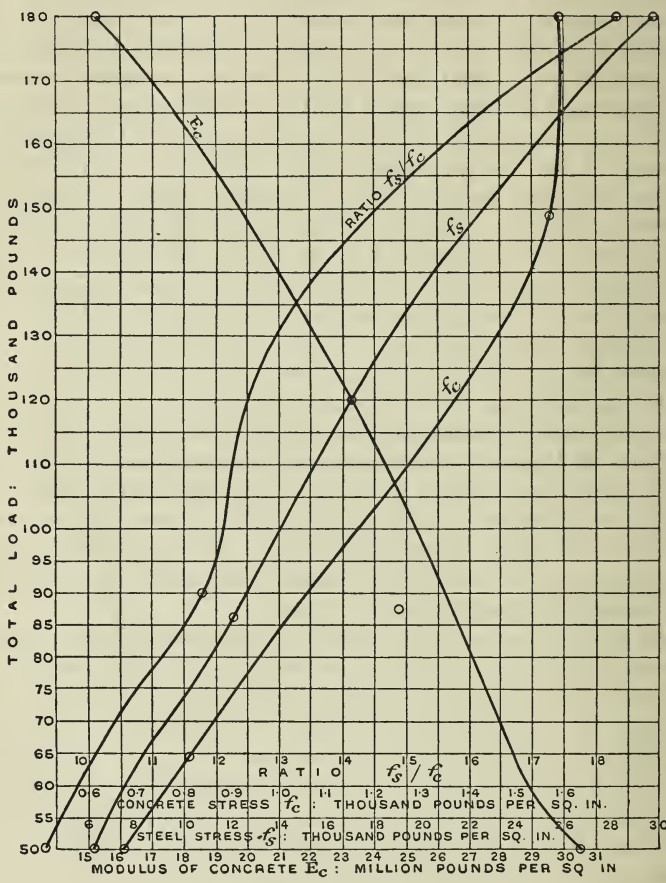
INCHES 12 6 0 1 FOOT

admirably to this plan and virtually indicated such a treatment. A structure designed on the foregoing lines would certainly cost more than one of the ordinary type of rod reinforcement, but there would be distinct gains in rigidity, safety, and reliability. The weight of steel in the girders and beams would be considerably greater than was generally used with tension-rod reinforcement (perhaps 75 to 90 per cent.), with a possible saving in the quantity of concrete too small to be important. The resulting construction would be, in his opinion, however, about the best available, at a cost 10 to 15 per cent. lower than that of the steel-frame and terra-cotta type. With care in adjustments, it might be possible to gain some advantage by subjecting the steel to construction loads or dead loads before and during the placing of the concrete, but this was a point requiring very careful consideration. Mr. Buel.

Mr. E. P. Goodrich, of New York, was much interested in the subject of Mr. Popplewell's Paper, believing that such direct experimental investigations were worth considerably more than theory in the correct design of reinforced-concrete structures. He felt, therefore, that it was unfortunate that the experiments had been conducted with a test-column designed as described. Theoretically, there must be exactly the same compression in steel and concrete at the centre of the test-specimen, which condition, however, would not be theoretically possible near the ends, provided the rods were stopped short of the end of the specimen, as was done in this instance. With the usual methods of design, the stresses in the steel were developed through the frictional resistance between it and the concrete, so that the general line along which the experiment had been conducted should be a very instructive one. Had the test points been near the ends of the rods, he believed that remarkably different results would have been obtained, those actually reported being, he thought, of small value under the circumstances. Grave doubt existed in his mind with regard to the experiments made to determine Young's modulus of elasticity, the results being hardly more than half of those which he would expect. This was clearly indicated when the results obtained were compared with those described in Mr. Burr's Paper. With stresses of approximately 600 lbs. per square inch in the concrete, the modulus of elasticity was over 3,000,000 lbs. per square inch. He therefore strongly recommended that the experiment should be repeated with new test-pieces, and with measurements so made as to determine the compression in the rods near the ends instead of at the centre. Further, that other elasticity tests should be carried out for the determination of Young's modulus. Mr. Goodrich.

Mr. Goodrich. With regard to Mr. Burr's Paper, it so happened that Mr. Goodrich had been connected with the design and construction of the 39th Street Building referred to, and had given a considerable amount of study to the design of concrete columns of the type described. That type had many advantages, some of which had been mentioned; but

Fig. 5.



it had one considerable drawback in that it was not as economical as was either of two alternative types. Each of the latter, however, might possess drawbacks which more than compensated for the economy in first cost. Estimates were made, based upon the use of ordinary reinforced columns, with spiral and longitudinal rods, for the building in question, and it was found that they would have

enabled several thousand dollars to be saved. Subsequent analyses on the part of several engineers showed that an ordinary steel column properly fire-proofed could have been obtained with a similar saving. Another item of interest to him was the Table showing the unit stresses in steel and in concrete. He had extended the figures down so as to include a 50,000-lb. loading for Columns Nos. 3 and 4, with the result shown in *Fig. 5*. It was evident from these curves that a marked falling-off occurred in the modulus of elasticity of the concrete with increase of stress. The results of the test showed that the unit stress in the concrete increased at a practically uniform rate up to about 1,500 lbs. per square inch, after which it increased less rapidly, and finally came to a maximum at about 1,600 lbs. per square inch. The corresponding stresses in the steel were seen to take the opposite tendency, namely, to increase so that the steel took a large proportion of the load after the concrete had been stressed to about 1,500 lbs. per square inch. At the lower stresses, such as were used for actual designing, the relative loads carried by the concrete and steel were quite different from those usually assumed. At a 750-lb. unit stress in the concrete, for example, the stress in the steel was only slightly over 7,000 lbs., the ratio of the moduli being seen to be less then 10. This was a new experimental demonstration of results obtained by Professor E. Mörsch[1]—for example, on reinforced-concrete columns of the usual type, in which it was found that the ratio of the moduli should be assumed at approximately 10 in designing columns, instead of 12, 15, or higher, as was usual. The higher value might be quite proper for reinforced-concrete slabs, beams, or girders, but was found to be inapplicable to columns.

Mr. Goodrich.

Mr. E. W. Hollingworth found Mr. Popplewell's Paper of much interest, giving as it did direct measurements of the stresses actually present in the steel and concrete, and throwing light upon the question of the slipping of the bars. On theoretical grounds it would appear desirable that the load should be applied directly to the ends of the reinforcing bars instead of being transmitted through the concrete. If the last inch of a bar in a column reinforced with bars free at the ends were considered, it was obvious that no stress occurred at the end of the bar. At 1 inch away from it there was only the stress which could be communicated by the bond on a 1-inch length of the bar, although at that point there was the full load upon the concrete. The concrete, therefore, would be compressed, while the steel was practically unstrained, with the

Mr. Hollingworth.

[1] "Concrete-Steel Construction," p. 60. New York, 1909.

Mr. Hollingworth.

result that there must be relative movement between the two. When the load was removed the concrete would return to its original length, again slipping on the bar. It would seem, therefore, that after a number of repetitions of the loading, the constant slipping of the bar would wear out the bond at the end of the bar. The same action would then extend to the next length, the destruction of the bond gradually extending along the bar as the load was repeated. The effect of failure of the bond was to throw the whole load upon the concrete, and a column reinforced with bars free at the ends would therefore be expected to fail under repeated applications of a load which would crush a cube of unreinforced concrete of the same sectional area as the column. This action would be inevitable if the stress were uniform over the cross section of the concrete, but the presence of the bars would prevent this from being the case, and it was possible that the elastic properties of the concrete might be sufficient to allow a relative motion to take place without breaking the bond. This could only be established by tests on columns under many repetitions of the loading. Observations taken at the centre of a column—the point at which the tendency of the bars to slip in the concrete was a minimum, and under loads applied two or three times—were valuable in themselves, but they were hardly sufficient evidence to justify the conclusion that there was no movement between the steel and the concrete in the column.

Mr. Leitch.

Mr. D. C. Leitch, while agreeing with Mr. Ellis's view that the foundations described constituted the only real difficulty in connection with the work at Shanghai, and congratulating him on having nevertheless constructed thoroughly stable and efficient works, ventured to think that some modification in the wharf-foundations would have been advantageous. Each group of four piles, supported mainly if not wholly by ground-friction, had to carry a maximum load of only 11 tons per pile. The piles were of ample strength to carry four times that load. It was suggested that as they could not be driven hard, less reinforcement would have sufficed, and that to increase the frictional hold they might have been either moulded with surfaces roughened or corrugated, or made triangular in form. For instance, triangular piles having 19-inch sides, with one bar near each angle, would have about 2 per cent. more frictional surface per lineal foot than those used, with only 78 per cent. of the volume of concrete, and 75 per cent. of the reinforcement actually used. As the pile-heads were embedded in the caps, the triangular form, usually inconvenient on account of connections, would occasion no trouble. But in deep alluvial soil, such as that described, if it was free from boulders, trunks of trees, etc., cylinder foundations were generally

more economical and satisfactory than piles. In this case cylinders of 4 feet 3 inches external and 3 feet 5 inches internal diameter, composed of 1 : 2½ : 5 concrete and reinforced by a spiral iron rod ½ inch in diameter with a pitch of 6 inches, and four vertical ⅝-inch rods, were suggested as preferable to the groups of four piles. They would, it was thought, be less costly, would resist lateral displacement better, would be less liable to irregular settlement, and less dependent on the reinforcement, and therefore would probably be more durable. Their supporting-power could be increased to a considerable degree after sinking by plugging them with concrete or filling them with gravel. Larger cylinders, say 6 feet in diameter, might be used where groups of six piles were shown. The diagonal longitudinal bracing extended over three bays only from the face, but the diagonal cross bracing was carried the full width of the wharf. Three bays from the face seemed sufficient, as the deck and horizontal waling took the thrust of vessels alongside. No doubt the diagonal bracing helped to distribute the load, but with better foundations this would not be necessary; it was not a satisfactory feature. At the quay-wall, the cracks 40 feet back from the face indicated a slip of ground on a slope of about 2 to 1. But even at 1½ to 1 the plane of slipping would pass clear of the deck, which did not thus appear to add to the stability of the wall in any material degree. Along with the counterforts, it served a useful purpose in stiffening the wall against the pull of the anchorage-ties, which were, no doubt, its main support. The tie-bars themselves could safely support a tension of 40 tons, and it seemed quite possible that this figure might be reached. But the splices appeared weak; allowing 100 lbs. adhesion per square inch they were equal to taking only 6 tons each. The end connections, made by hooking over the ends of the 1⅜-inch bars, appeared weaker still. At the inner ends, where they were encased in plastic material which could not resist the straightening of the hook under tension, this might not be important, because, no doubt, a considerable part of the tension was taken up by the frictional hold of the earth on the concrete casing of the tie. But at the lower end something more seemed desirable; it would have been well to continue the ties into or through the deck. The plastic composition at the inner ends of ties could hardly afford any relief if the building-foundation moved vertically, as it had done. Concrete might as well have been used, or, to provide against injury to the ties due to settlement, a short length of heavy chain (say made of iron 2 inches in diameter), carried well back into the foundation, and embedded for about 4 feet in the concrete between the 1⅜-inch bars. Assuming that the British standard specification

Mr. Leitch.

Mr. Leitch. provided for carrying the grinding of Portland cement to the utmost limit compatible with economy under ordinary conditions of use in England, it appeared that, where transport-charges largely increased the cost of cement, some modification might be advisable. It was generally admitted that the residue on a 180-mesh sieve had little cementitious value. Inert material could always be found at the place of use. In the Transvaal, where cement cost about £7 per ton, it was economical to stipulate for a residue not exceeding 10 per cent. on the 180-mesh sieve, and there was no difficulty in getting this. Such cement, though it set when neat with rather inconvenient rapidity, gave no trouble on this score when mixed with sand as, except for testing, it always was. The cost at Shanghai was not stated, but was probably high enough to render fine grinding advisable. The most economical proportions for concrete depended of course on the relative cost of cement and of the other items; and generally, where the weight of the concrete formed a large portion of the load, as in beams or decking, richer mixtures were advisable than where, as in columns and foundations, it did not do so. In the present case the mixtures seemed to be needlessly rich, and Mr. Leitch thought that, with cement ground to leave only 10 per cent. on the 180-mesh sieve, the following would have sufficed: foundations, 1 : 3 : 6; piles and caps, 1 : 2½ : 5; decking, beams and bracing, 1 : 2¼ : 4½. He had used 1 : 2 : 5 concrete, with 0·5 to 1 per cent. reinforcement, extensively, even for arches, pipes, and cylinders 2½ to 3 inches in thickness, and had found the results satisfactory. There was complete adhesion to the reinforcement and freedom from interstices. With native labour and inadequate supervision a larger proportion of mortar in the concrete might be necessary.

Mr. Marchbanks. Mr. JAMES MARCHBANKS observed that the time was fast approaching when engineers in New Zealand would be compelled to adopt some form of concrete construction for large wharves, because every year it was becoming more difficult to obtain first-class timber, such as New South Wales ironbark, at a fair price and in sufficient quantities, in a reasonable time. He proposed to adopt reinforced concrete in the next large wharf for ocean-going steamers to be built at the port of Wellington. The bottom on the site of the wharf which Mr. Ellis had constructed at Shanghai appeared to be very bad indeed, but the method adopted by him of driving groups of piles connected together about mid-tide level, while increasing the bearing-area to take the required loading, did not unduly add to the work in the superstructure. It would be interesting to know if there had been any great difficulty in getting a good solid connection between the heads of the piles and the

lower walings when they were placed in position, and whether any contraction cracks, due to shrinkage of the green concrete where it abutted on these walings, had taken place. The quay-wall appeared to have been built in one continuous length. Had any shrinkage cracks appeared in it near the coping? Judged by New Zealand conditions the cost of the work was extremely low. A reinforced-concrete wharf built at Wellington in 1909 cost 16*s.* 6*d.* per square foot. This, however, included wood-blocking the surface with hardwood blocks and driving timber spring piling with longitudinal fenders outside the reinforced-concrete piles, which cost £5 12*s.* 3*d.* per lineal foot of frontage. Mr. Marchbanks.

Mr. A. H. Ough had found that in Hong-Kong supervision of reinforced-concrete work had to be practically continuous, and was very costly. It would be interesting to know if the costs given by Mr. Ellis on p. 20 included supervision; if not, what the total cost of supervision was. In 1906 he commenced reinforced-concrete foundations for "godowns" in Hong-Kong for Messrs. Alfred Holt. During 1906, 1907, and 1908 he also completed two blocks of four-story godowns and wharves for them, somewhat similar to those described in the Paper. Mr. Ellis, as their consulting engineer, inspected the work on several occasions during progress. In this work the godowns were steel-framed, with brick panels, and the wharves were constructed of steel Differdange piles with channel bracing. In the principal wharf the whole structure above low-water level was cased with 2 inches of concrete. There was 30 feet of water alongside at low water. The main bearing-piles were enclosed in steel cylinders, which were afterwards filled with concrete deposited under water by hoppers. Considerable difficulty was experienced in getting the booms and bracing properly covered by the concrete, as very little time was available at low water while the sea was smooth, and in places where the waves had washed over green concrete the moulds stripped badly, and concrete had to be cut out and replaced. The total cost of the wharves, exclusive of supervision, was $560,000, say £49,450, or 16*s.* 3*d.* per superficial foot of deck. The cost of European supervision was 9*d.* per superficial foot of deck. The largest class of "Blue Funnel" steamers berthed alongside this wharf, and after 3 years' use no cracks had appeared in the concrete protection. A slight elasticity was, however, perceptible in the structure when a heavy ship bore against it. The wharf was designed to carry a standard-gauge railway-siding, and cargo weighing up to 50 tons per piece. In 1908 he used groups of concrete piles for foundations of case-oil storage-godowns, constructed for the Standard Oil Company. The foundation was Mr. Ough.

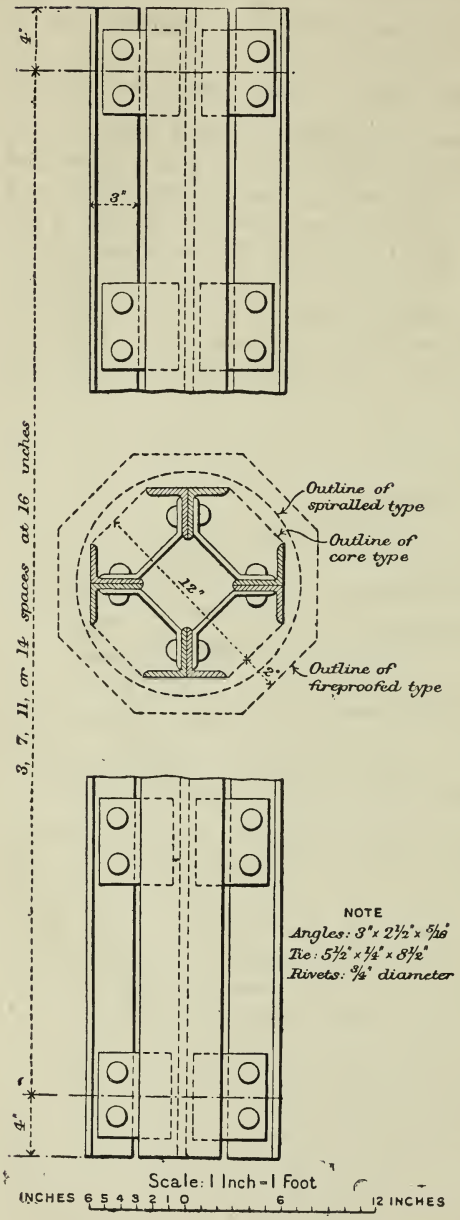
Mr. Ough. soft earth filling on 25 feet of mud, over a varying substratum of hard decomposed granite and rock. For lengths up to 58 feet a steel tube 10 inches in diameter, shod with a loose conical cast-iron plug, was employed, and was driven by a 3,000-lb. monkey, operated by a Lidgerwood hoisting-engine. Driving continued until a maximum set of $1\frac{1}{2}$ inch was obtained with the monkey falling 6 feet. A short length of tube filled with sand and plugged with hardwood was used as a dolly to transmit the blow of the monkey to the tube. Hand-mixed concrete in the proportions 1 : 2 : 4 was poured into the tube to a height of 3 feet; a rammer was then inserted on top of the concrete, and the monkey was allowed to rest on it, while the tube was lifted 2 feet 6 inches. The rammer was withdrawn, another 3 feet of concrete was poured in, and the operation was repeated until the surface of the ground was reached. Concrete shoes and hardwood shoes were tried, but were abandoned in favour of cast iron. A rectangular slab of concrete 1 foot thick, reinforced with a grill of 1-inch bars, connected the pile-heads in each group of four or five piles; these heads carried reinforced beams on which the walls were built. A typhoon occurred soon after the piles and slabs were completed, and the sea washed out the filling from many groups of piles, leaving them standing 6 to 10 feet above the surface of the ground, without injury. The filling was replaced and the reinforced beams and buildings were proceeded with and completed. At present the only settlement that had taken place was at one corner, where the piles were not got down to the hard rock, the steel tube having been driven to refusal at a depth of 58 feet. The filling at this spot, being immediately behind the sea-wall and wet, had settled bodily 6 inches, taking the piles down with it, but the resulting cracks in the building were not serious. Where the piles were supported in dry filling by skin-friction only, no settlement had taken place. The actual load borne by each pile was 8 tons. The pile formed by the 10-inch tube was 13 inches in diameter. The cost of piles, including the upkeep of the piling-engine, coal, stores, labour, and material, but exclusive of constant European supervision, was 75 cents (15·75*d.*) per cubic foot of concrete, measured on the finished pile. Supervision cost 5 cents, or just over 1*d.*, per cubic foot. The piles were put in at the rate of 200 lineal feet per working-day of $8\frac{1}{2}$ hours. In a large number of flat roofs, verandah floors and wharf-decks in Hong-Kong Mr. Ough had used expanded metal and concrete. Spans of 8 to 13 feet were employed, with a total thickness of 4 to 6 inches of concrete. The expanded metal was laid on wood horsing, and the concrete, mixed 1 : 3 : 5,

was deposited on it and brought up to a face with the shovel. It was well chopped into the mesh, causing the metal to rise 1 inch into the mass. On removal after 14 days the horsing left a clean surface. The stresses due to contrary flexure over bearings were taken by strips of expanded metal, bedded in the concrete 2 inches below the upper surface. In 1911 the deck of a small wharf constructed in 1908 in this manner was burst up by the sea during a typhoon. The concrete failed by splitting on the plane of the reinforcement, and the expanded metal had rusted badly and separated from the concrete. A concrete cover of 2 to 2½ inches was necessary to prevent corrosion in reinforced concrete exposed to sea-water in that climate. In reinforced-concrete roofs expansion-joints were necessary about 12 feet apart. Mr. Ough.

Professor A. N. Talbot, of the University of Illinois, thought that the tests of composite columns reported by Mr. Burr threw light on the action of a form of column which had been used frequently in recent years. Doubt had existed in the minds of many engineers as to the division of stress in such columns between the structural steel and the concrete. Such tests, therefore, would be welcomed by constructors. Equation No. 1 (p. 48) seemed to be based on the use of a constant modulus of elasticity for the steel. In *Figs.* 2 and 4 (pp. 44 and 46) more shortening was shown at loads above 100,000 lbs. than would be obtained by the use of a constant modulus. If for any given amount of shortening in the composite column the load carried by the plain steel column for the same shortening were taken as the load carried by the steel, the values obtained by this method for the stresses in the steel would be smaller than those given in the Table on p. 49. The stresses carried by the concrete, therefore, would be found to be higher, by 15 or 20 per cent. for the higher loads, than those given in the Table. Of course, the modulus of elasticity of the concrete obtained by this method would be higher than the values given by Mr. Burr. In 1911 tests of thirty-two columns, somewhat similar in form to those described in Mr. Burr's Paper, were made at the University of Illinois in an endeavour to determine the effect of length of column and the portions of the load taken by the steel and the concrete. The tests included:—Plain steel columns; columns of the "core" type, with the interior space filled with concrete (the portion of a column which would be considered in designing); columns with a 2-inch shell of concrete around the structural shapes (called the "fire-proofed" type); and columns having spiral hooping on the outside of the structural shapes. The general form of the sections was shown in *Figs.* 6. The steel column was known as Prof. Talbot.

Prof. Talbot.

Figs. 6.



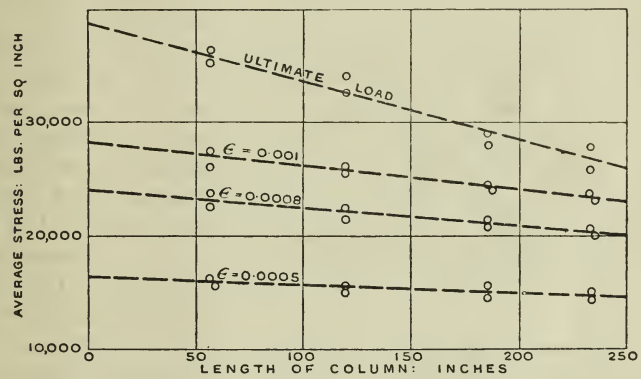
the Gray column, and was composed of angles tied together at intervals by bent plates. The area of the steel section was 13 square inches, with a radius of gyration of 3·9 inches. The length of column used was generally 10 feet, but for the plain steel columns and the core type of column, lengths of 4 feet 8 inches, 15 feet 4 inches, and 19 feet 4 inches were also tested. The concrete was composed of 1 part of cement, 2 parts of sand, and 4 parts of broken limestone, measured by loose volume, except that columns with 1 : 1 : 2 and 1 : 3 : 6 concrete were also made. The maximum loads carried by the plain steel columns were given by the equation :

$$\frac{P}{A} = 36{,}500 - 155\,\frac{l}{r},$$

where $\frac{l}{r}$ was the ratio of the length of the column to the radius of gyration, and the units were pounds and inches. At smaller loads the effect of length was considerably less, the relation being expressed by the following equations for three values of

unit-shortening of the column: For a unit-shortening (ϵ) of 0·0005, Prof. Talbot.
$\frac{P}{A} = 15{,}750 - 27\frac{l}{r}$; for $\epsilon = 0{\cdot}0008$, $\frac{P}{A} = 23{,}500 - 55\frac{l}{r}$;
for $\epsilon = 0{\cdot}0010$, $\frac{P}{A} = 28{,}500 - 82\frac{l}{r}$. These stresses for different lengths of plastic steel column were plotted in *Fig. 7*. For the concreted columns of the core type, the effect of length on the total load carried by the column approximated very closely to the effect found in the plain steel columns; in other words, the load carried by the concrete was nearly the same for all the columns, irrespective of length. The stress taken by the concrete of the composite columns, determined by deducting from the total load on the column an amount equal to the load carried by the plain steel columns of the same length at the same unit shortening, was about two-

Fig. 7.



thirds of the strength of cubes made of the same concrete. Values of the ratio of steel stress to concrete stress (m), which seem to be applicable to such columns, might be given as 16 for the 1 : 1 : 2 concrete used, 25 for the 1 : 2 : 4, and 35 for the 1 : 3 : 6. A basis for design which seemed rational in columns in which the structural steel shapes constituted 8 per cent. or more of the area of the column was the following:—First, the strength of the steel column was determined by the use of the column formula for the l/r of the steel column section; secondly, the concrete of the core section (without reference to the length of the column for any ordinary length-ratio, say, up to a length of 15 diameters) was considered to have a stress value proportional to the strength of plain concrete, say, two-thirds of the cube strength. A suitable factor of safety, of course, would be applied somewhere.

Prof. Talbot. In the tests, the columns of the core type showed considerable toughness, though at the maximum load there was little lateral deflection. In the tests of the fireproofed type of column (which had a shell of concrete outside the steel) the concrete shell remained intact until a shortening was reached as great as that developed at the maximum load in columns of the core type. This integrity of section at high deformations indicated that the presence of the shell need not impose any restrictions upon the working-stresses available for the steel and for the core concrete. Of course, there were good reasons for the use of a metal binder, in the form of a wire mesh or spiral, for holding the shell securely in place. The results indicated that the stress carried by the concrete shell was much less than that carried by the concrete core. The action of the spiralled columns showed that the spiral had little effect up to a deformation and load corresponding with the maximum load for an unspiralled column. Beyond this load the column compressed rapidly, and though the presence of the spiral added materially to the strength of the column, it was not felt that this additional strength could be considered as available to any great extent in the design of buildings. The composite columns tested showed the qualities of a good structural member, and this form seemed well adapted to more general use in building construction. Where the structural shapes constituted a considerable part of the carrying strength of the column (say, 8 per cent. or more of the total column section) the structural steel portion might well be considered as the basis of design, and the working-stress for the steel defined by the usual methods. The stress to be used in the concrete would be related to the stress in the steel. In order to ensure safe values for the strength of such columns designed on this basis, a low modulus of elasticity of the concrete must be selected. With this in view, and judging from the tests referred to, it would seem that the value of the modulus of elasticity of the concrete should be taken at much less than the 2,000,000 lbs. per square inch given by Mr. Burr for ordinary concrete, in order to be on the safe side. In conclusion, Professor Talbot felt that Mr. Burr's strictures on columns made with longitudinal steel rods as reinforcement were not altogether well founded, although he agreed that much care should be taken in the design and construction of such columns.

Mr. Trautwine. Mr. J. C. TRAUTWINE, Jun., of Philadelphia, pointed out that Fig. 2, Plate 1, indicated that the piles had been driven, in general, but little more than 25 feet, so that for the greater part of their driven length they should have been in "the top layer of ground, averaging 25 feet in thickness." This, it was mentioned on

p. 6, might usually be reckoned upon to form a solid crust of stiff, sandy clay, though rendered irregular on its surface by the presence of old creek-beds, filled with softer mud; but in discussing the behaviour of the piles Mr. Ellis referred to them as being in mud. His account of their behaviour added to the long list of cases which strikingly illustrated the fact that the resistance of piles in mud to loading was ordinarily much greater than might be inferred from their very slight resistance to driving. Piles which had sunk under their own weight alone often resisted hard blows or heavy loading when time had been given for the conditions of driving to be superseded by those obtaining after the material had come to rest. Mr. Ellis plausibly attributed this difference to "the film of water," between the mud and the pile, which, he concluded, had time to escape after driving and before loading, the conditions of true skin-friction being thereby established. By comparing many recorded tests of driven piles, Mr. Trautwine had found that a co-efficient involved, which seldom exceeded 0·3 in hard ground, was much greater in soft mud, sometimes exceeding unity. This experimental coefficient (c) might be expressed in the equation Mr. Trautwine.

$$R_l = c\,R_d = c\,\frac{W\,h}{s};$$

where R_l was the ultimate resistance of one pile to loading, R_d its resistance to driving, W the weight of hammer, h the height of fall, and s the set. In other words, the change of condition, in mud, during the time intervening between driving and loading, was so great that the pile might resist a load greater than even the theoretical ultimate load, or resistance to driving, represented by $W\,h/s$; the value of c increasing progressively, as the ground became softer. From this it resulted that the resistance to loading was not a constant function of the resistance to driving. The data studied indicated that R_l was more nearly a constant function of $\sqrt{R_d}$; that was to say, in the equation $R_l = k\sqrt{R_d}$, m was nearly independent of the value of $W\,h/s$. The factor k varied, with other conditions, generally between 100 and 200; so that c ranged, in general, between the values $c = 100/\sqrt{R_d}$ and $c = 200/\sqrt{R_d}$. In the present case,

W = 3·5 tons = 7,840 lbs.;
h = 6 to 8 inches;
s = generally 1 inch;

whence

$$R_d = \frac{W\,h}{s}$$
$$= 47{,}040 \text{ to } 62{,}720 \text{ lbs.};$$

Mr. Trautwine. and the ultimate resistance to loading ranged

from $R_l = 100 \sqrt{R_d} = 21{,}700$ to 25,000 lbs.
to $R_l = 200 \sqrt{R_d} = 43{,}400$ to 50,000 lbs.

The actual maximum working-load was stated as 45 tons (100,800 lbs.) on four piles, or 25,200 lbs. per pile ($c = 0{\cdot}4$ to $0{\cdot}53$; $k = 101$ to 116), indicating a safety-factor of, say, 1 or 2, as compared with ultimate loads found by the foregoing formulas. Under this load, however, one of the piers settled $1{\cdot}75$ inch in a fortnight and then stopped, but settled $1{\cdot}5$ inch more after lengthening and re-driving the piles; indicating that the working-load on the piers in general was not very much less than the load which would cause settlement. In selecting the factor of safety in pile-driving, it would seem proper to consider chiefly not (as usual) the character of the ground (provided this might be assumed to be constant) but rather that of the proposed structure, and the uses to which it was to be put. Thus, while a factor of safety of 2 would be far too small for an American "sky-scraper," or for a small-span bridge carrying heavy trains, it might be sufficient for Mr. Ellis's case of a wharf, which carried buildings of no great height, presumably subject to no relatively heavy moving loads, and capable of enduring slight settlements without serious damage. The formulas for pile-resistance in use hitherto had been based upon a priori reasoning rather than upon the records of actual practice; and engineers had regarded them with so little confidence that excessive factors of safety had been in common use. Weisbach mentioned factors ranging from 10 to 100 (!) as being usual. But Mr. Trautwine's study of recorded cases indicated that, by the use of these formulas (with coefficients derived from the cases studied), in cases like those at Shanghai the ultimate load might generally be estimated with sufficient confidence to warrant the use of factors of safety of 2 to 4; provided the condition of the ground might be taken as not liable to change after loading.

Mr. J. S. E. de Vesian. Mr. J. S. E. de Vesian felt that the experimental results and conclusions stated by Mr. Popplewell were opportune as a reminder of the fact that the natural bond of concrete and steel, when properly applied in combination, required no mechanical or adventitious aid of the kind suggested by advocates of patent bars with projections of different forms. This had been established also by long practical experience, and the investigations of Bach, Bauschinger, Ritter, Hatt, Talbot and others. Mr. Popplewell's Paper showed clearly that in the columns tested, which were reinforced by plain round bars, there was no evidence

Mr. J. S. E. de Vesian.

of movement of the concrete relatively to the steel, and as Mr. Popplewell stated, the conclusion that no slipping was taking place was supported independently by the experiments on frictional grip. The Table of frictional resistance to slipping (p. 36) denoted very satisfactory values for concrete of the low compressive resistance stated in the Table on p. 27. It justified the inference that the values would be considerably higher for concrete of the same proportions, but prepared so as to give greater compressive strength. Owing to the small number of test-prisms and to variations of manipulation in their preparation, the results were not particularly consistent, and did not indicate in any way the increase of adhesive power that took place with increase of age. The investigation made under the direction of Mr. Mesnager in 1903[1] for the French Government Commission on Reinforced Concrete showed that the resistance of concrete and steel to slipping depended very largely on the quality of the concrete and particularly upon the age of the combination of the two materials. As bearing out the latter point, the results given on p. 107 of the Report made by the Commission might usefully be quoted. The experiments in question were made upon pieces cut from a sleeper which had been in service for several years in the permanent way of the Western Railway of France. The concrete was reinforced by rods 5 millimetres (about $\frac{3}{16}$ inch) in diameter. Seven specimens were tested, with the following results:—

No.	Resistance to Slipping.		Observations.
	Kg. per Sq. Cm.	Lbs. per Sq. In.	
1	41·6	592	Steel broke outside the concrete.
2	50·0 1st trial	711	No slip under the load applied.
	81·2 2nd trial	1,155	Steel slipped.
3	74·0	1,053	Steel broke outside the concrete.
4	87·1	1,239	Steel slipped.
5	92·0	1,308	Steel broke outside the concrete.
6	81·3	1,156	Steel slipped.
7	57·6	819	Steel slipped.

The high values shown above were undoubtedly due in part to the fact that the rods embedded were not perfectly straight. This was a matter of importance, as in actual practice the reinforcing

[1] "Commission du Ciment Armé: Expériences, Rapports...relatives à l'emploi du Beton armé," p. 104, Paris, 1907.

Mr J. S. E. de Vesian.

bars were never perfectly straight, and in the case of beams they were very often bent purposely. The columns tested by Mr. Popplewell were generally of good design, although the lateral links were spaced farther apart than Mr. de Vesian considered advisable. This, however, was not important so far as the tests were concerned, for, as stated by Mr. Popplewell, the presence or absence of lateral tie-bars did not affect the calculations. It should be noted also that the quality of the concrete was distinctly poor, in spite of the satisfactory proportions adopted. Concrete of the same proportions used by Mr. de Vesian on the General Post Office buildings and elsewhere had developed an ultimate compressive strength of about 3,550 lbs. per square inch at the age of 6 months, or nearly double that exhibited by Mr. Popplewell's test-cubes at the same age. The discrepancy was probably due in part to the use of Leighton Buzzard sand, which, although useful for standard tests, did not comply with the conditions necessary for strong concrete. Another important factor, of course, was the manner in which concrete was mixed, his own experience being that proper mixing with approved machinery resulted in a considerable augmentation of compressive strength. While the low quality of the concrete did not invalidate in any way the conclusions of the Paper as to the security of the bond under the small loads applied, it certainly must be taken into account in considering the relatively small values given for frictional resistance on p. 36. The results obtained from concentrated loading entirely agreed with Mr. de Vesian's experience. If the concrete was rather more compressible than it should be, the steel helped it, and took slightly more than its proper share of the load. This mutual help of the component parts was one of the great advantages of reinforced concrete. Mr. François Hennebique had cited this material as an illustration of what a perfect republic should be, the strong helping the weak and all working together to bear the common load. The fact, to which Mr. Popplewell drew attention, that the grip of concrete on steel in tension could not be maintained for long after the steel had begun to yield, was borne out by experience. The ideal steel for reinforced-concrete work would be one with a high elastic limit coupled with a satisfactory extensibility, that was to say, a low-carbon steel with a high elastic limit.

Mr. Waite.

Mr. Guy B. Waite mentioned that the experiments made by Mr. Burr were of unusual interest to him, as he had borne the responsibility for constructing the columns tested, and had been present during the tests. The riveting of columns Nos. 1–4 was done by hand. The $\frac{1}{2}$-inch bolts of columns Nos. 5 and 6 were fitted rather

loosely, about as would be done in actual practice. There should be no surprise that columns No. 5 and 6 gave poorer results than Nos. 1 and 2; the latter were latticed, and thus braced against side flexure, while Nos. 5 and 6 were substantially 3-inch independent channels. Further, the individual angles of columns Nos. 1 and 2 had a smaller ratio of l to r than the channels of Nos. 5 and 6. It should be noted that no side flexure of consequence took place in the failure of any of the composite columns. In each case the failure of the steel shell was due to bulging near the end; not a single rivet or bolt was disturbed. The $\frac{1}{4}$-inch batten-plates of columns Nos. 5–8 were easily bent up with a hand-bender, and could have exerted little resistance without the connecting bolts. Had the shear of the concrete exerted any great side pressure, some of the bolts ($\frac{1}{2}$-inch) would have indicated this force. The manner of failure of the empty steel columns was, to his mind, of little consequence; the loads that such columns would carry before the insertion of the concrete, was small. The tests showed that very little (probably green) concrete would brace the steel shell sufficiently against flexure to enable it to carry good working-loads. The ratio l/r for columns of this type in actual practice for ordinary buildings would be less than for these test-columns: therefore, with concentric loading, flexure could safely be ignored and full working-loads be allowed. Mr. Waite saw no reason for concluding from the tests that the latticed type of column was preferable to the batten-plate type; he was convinced that with uprights having the same value of l/r, the one latticed and the other with battens, they would give equal results on test. He was so sure of this that he was using the latter type and saving the unnecessary expense of latticing. From experience with more than one hundred steel-and-concrete buildings, he believed that tests on concrete 3 months old afforded a conservative estimate of its strength, as in practice a longer time elapsed before the concrete was required to carry much loading. He had used this form of column in several buildings, and had found it very convenient, in that the building could progress without waiting for the concrete to set. The heaviest service to which Mr. Waite had subjected this form of column was in an eight-story warehouse in New York City, where the total floor-load was 350 lbs. per square foot with a floor-area of about 300 square feet bearing on the columns from each floor. A heavier service, under these conditions, would not be practicable (without excessive steel), on account of the large size of the columns.

Mr. Waite.

Professor W. H. Warren thought that Mr. Popplewell's determination of the deformations of the steel rods and of the concrete

Prof. Warren.

Prof. Warren. separately proved clearly that the two materials might be relied upon to work together. In good reinforced-concrete columns they would divide the load in proportion to their areas and to the values of the coefficients of elasticity. There was, however, a disturbing element due to the shrinkage of the concrete when hardened in air, as this would put initial compression upon the rods and might reduce their efficiency in carrying a proper share of the load. In his own experiments, made in 1904, various types of reinforced-concrete column were investigated,[1] and the efficiency of the spiral band or hooping was shown to be more than twice as much as that of the longitudinal rods of the same weight. This confirmed Mr. Considère's investigations on the efficiency of spiral hooping. It was quite clear, however, that the spiral hooping or the transverse ties assisted the concrete by preventing lateral swelling, and were in tension. This action therefore could not be developed with moderate stresses such as would be considered safe for concrete, and it followed that concrete reinforced in this way must behave very like unreinforced concrete when subjected to ordinary safe working-stresses. The spiral hoop however provided an additional security against ultimate failure, and when combined with longitudinal rods of about equal weight it formed a very efficient reinforcement for concrete piles and columns. In the Paper by Mr. Ellis there were some good examples of reinforcement by means of spiral bands; but in the calculation of the safe load on the experimental columns the real action of the spiral was ignored, although the result of the calculation was approximately correct. It appeared to Professor Warren that the experiments on the unreinforced columns indicated a safe compressive stress of 700 to 800 lbs. per square inch for such concrete—say 750 lbs. Allowing 12,000 lbs. per square inch for the steel, an equivalent area would be obtained for the rods and hooping of 5·2 square inches (equivalent to $2{\cdot}4 \times 0{\cdot}69 = 1{\cdot}66$ square inch). The area of the octagon less the area of the reinforcement was about 75 square inches, and the area of the core was 46·74 square inches, so that the safe load was

$$\begin{aligned} P &= (46{\cdot}74 \times 750) + (5{\cdot}2 \times 12{,}000) \\ &= 97{,}455 \text{ pounds, or } 43{\cdot}5 \text{ tons.} \end{aligned}$$

This neglected the area of concrete outside the core, but, up to the point where this flaked off, the whole area would contribute to the carrying-capacity of the column, and the safe load would be

[1] "Further Experiments on the Strength and Elasticity of Reinforced Concrete," Proc. Royal Society of New South Wales, vol. xxxviii, p. 140.

52·5 tons. With regard to Mr. Burr's Paper, setting forth the advantages of the steel column with a concrete core, as compared with a concrete column reinforced with longitudinal rods and spiral bands or transverse ties, there was no doubt that the steel would effectively assist the carrying-capacity of the column whether the stresses developed were small or great, and there it also had an advantage in regard to the design of the details of connections. Professor Warren had just commenced a new series of tests on concrete columns reinforced in various ways, but the work was not sufficiently complete to quote results. Prof. Warren.

Mr. Ellis, in reply, agreed with Mr. Brenan that a complete and strongly-designed system of timber fenders was required to protect the face of a reinforced-concrete wharf. In the structure in question, the fender-piles were fastened to the concrete by heavy steel straps, as being more secure than through bolts, and were connected by two rows of 12-inch by 12-inch walings. He also agreed with Mr. Leitch that in the interior groups of piles economy might have been safely effected by reducing the size of the reinforcing rods. The distress likely to be caused to the piles in driving was over-estimated in designing them; and by the time they came to be driven the greater number had been moulded and the skeletons prepared. It was very doubtful whether increasing the superficial area of the piles by altering their shape would have added to their bearing-capacity, as the ground would at the same time have been less compressed, and so more liable to subsidence in itself. For a similar reason cylinders had been regarded as out of the question, as with them no lateral compression of the ground would have been effected. He could not agree with Mr. Leitch that the diagonal cross bracing was an unsatisfactory feature of the wharf. With the liability to impact from 10,000-ton vessels on the face, and a tendency to pressure from an unstable shore at the back, the diagonal bracing might have been desirable across the full width of the wharf even had the foundations been good. Mr. Leitch deduced from the movement of the creek-wall, and from the fact that a crack appeared in the ground 40 feet behind it, that the deck of the wall was valueless for purposes of stability. It was, on the contrary, an essential feature of its design, forming, together with the weight of the filling above it, and the support of the piles below, a structure stable against overturning under the severest local conditions. The slight movement which took place before the anchor-ties were put in was entirely a forward one. Incomplete details of the tie-connections and splices were given in the Paper, and Mr. Leitch had assumed that they were weaker than the tension-bars them- Mr. Ellis.

Mr. Ellis. selves. It should, perhaps, have been explained that the precautions obviously necessary to make the joints of adequate strength had been taken. A length of chain might certainly have been used for the connections, but the device actually adopted appeared to answer equally well. Mr. Ellis had been much interested in Mr. Trautwine's views, which accorded with his own experience, as to the bearing-capacity of piles driven into silt. The description of the upper crust of ground as being of the nature of stiff, sandy clay, referred to the site of the warehouses rather than to the bed of the river, where the wharf-piles had been chiefly driven. With reference to Mr. Ough's question, the cost of European supervision on the part of the contractors was included in the unit figures given. That on the part of the engineer (excluding his own fees) amounted to nearly 4 per cent. of the total cost, as close inspection was exercised by a number of highly-paid men. The concrete piles used by Mr. Ough in the foundations of the Standard Oil Company's godowns appeared to be much the same as "Simplex" piles, which had been described[1] by Mr. Ellis in a Paper read before The Institution in 1907, the chief difference being that instead of the hinged "alligator" point to the "Simplex" steel form, a separate cast-iron shoe was used for each pile. Mr. Bergin had somewhat over-estimated the weights to be lifted, the heaviest member being the 50-foot face-piles, which weighed less than 5 tons. In common with other piles they were slung from hand-worked overhead travellers which ran on gantries along the moulding-yard and out on to a jetty. Here they were lowered direct on to pontoons, for conveyance to the pile-drivers. He agreed with Mr. Bergin that it would have been better to form contraction-joints in the deck of the wharf, but he could not join with him in thinking that such joints (in the deck surface alone) were incompatible with the design adopted, which was, from every other point of view, preferable to one entailing a series of self-contained units.

Mr. Popplewell. Mr. Popplewell, in reply, considered it quite possible that the Leighton Buzzard sand was responsible for the relatively low values given by the crushing-tests, as suggested by Mr. de Vesian, although he had frequently tested samples of concrete actually being put into work, with results much lower than those given in his Paper.

Mr. Burr. Mr. Burr, in reply, remarked that he had already observed essentially that he might concur, without prejudice to his own judgment, in the view put forward by Mr. Buel that the design of the columns

[1] Minutes of Proceedings Inst. C.E., vol. clxxi, p. 147.

tested scarcely appeared to exhibit such efficiency or economy as might reasonably be expected. The percentage of steel was not higher than, or even as high as, had been used in some spiral-wound or other similar banded columns; nor had effort been made to secure such high-grade concrete as might readily be obtained in laboratory work. Mr. Buel's proposal that in the so-called composite type of steel reinforcement for high buildings the steel section should be sufficient to carry the entire load of the building without stressing the steel beyond the elastic limit, ignoring all resistance of the concrete, had an advantage in one respect; but whether that system of design would secure reinforced-concrete columns of this type of greater or less capacity than would be designed in the ordinary way, would depend obviously upon the relative proportions of steel and concrete reached in the usual procedure of design for such columns, and could be determined only by actual computations. If concrete were not a dependable structural material it would appear to be advisable to omit it from construction; but probably few engineers would be willing to take that view of concrete at the present time. It seemed scarcely reasonable, therefore, to treat it as if it might become valueless at some future time, or under some extraordinary exigency. The high ultimate intensities quoted by Mr. Buel, both for plain concrete cylinders and for hooped columns of 1 : 2 : 4 concrete, should be received with some caution. The records of every laboratory in which much concrete had been tested would, it was true, show such high results after sufficient age in some cases, but they were generally due to exceptional conditions. Such values were certainly not usually reached in ordinary structural practice. The skilled and careful treatment given in the best laboratory work would lead to much higher values than might reasonably be expected as the product of even first-class labour in the field. There were many tests of spiral-wound or hooped columns which showed ultimate resistances of 1:2:4 concrete not exceeding 2,000 to 2,500 lbs. per square inch. Again, it was occasionally a matter of doubt what should be regarded as the ultimate resistance in some spiral-wound columns, especially when there was a high percentage of lateral reinforcement. The experiments of Professors Talbot and Withey, described in the bulletins of the Universities of Illinois and Wisconsin respectively, showed that the yield-point in such members was not greatly affected by the presence of the lateral reinforcement, although the latter, especially if it was represented by a high percentage, would hold the incipiently crushed concrete in condition to sustain a largely increased load. Obviously it would at times be difficult to determine precisely at what degree of deformation under

Mr. Burr.

Mr. Burr. such circumstances the ultimate resistance was to be taken. The column suggested by Mr. Buel's *Figs. 4* had some excellent points, but for long columns which must offer material resistance to bending the steel reinforcement had the defect of not being completely braced as a direct load-carrying steel member. In determining the comparative economy of steel and reinforced-concrete columns of the type under discussion, and considered by Mr. Goodrich, it was perhaps only fair to point out that the protection against corrosion and against the effects of an internal conflagration of stock or fittings should be taken into consideration. The first cost was, of course, generally the controlling factor in the discussion of the economy of high-building construction, but there were certainly other elements of the question which should be considered. There was doubtless, as Mr. Goodrich had pointed out, some uncertainty as to the distribution of load between the direct load-carrying steel reinforcement of this type of column and the concrete, which could only be cleared away by a series of tests much more extended than any now available. He was not sure that the method of attacking the question followed by Mr. Goodrich in this particular instance was without error, although his results were significant. It was probable that the law of distribution changed somewhat with the increase of loading, but so low a ratio between the moduli of elasticity of steel and concrete as 10 or 12 did not seem reasonable in view of present experimental data. Professor Talbot had cited some highly interesting results of his extended experience in the testing of reinforced-concrete columns, the general significance of which did not seem to be substantially different from what was indicated by Mr. Burr's four column-tests. In determining the relative amounts of loading sustained by the direct load-carrying steel reinforcement and the concrete, he suggested treating the two parts separately. It appeared to be a serious question whether the two parts of the same unit might thus be treated independently. Probably the most satisfactory results might be obtained by careful measurement of the actual compression of the steel, for as long lengths as practicable, under loads advanced progressively. If this procedure were followed, and if the modulus of elasticity of the steel were determined with accuracy, the actual load sustained by the steel would, of course, be disclosed. If this quantity were then subtracted from the total load carried by the reinforced column, the load taken by the concrete core would at once follow. From what he had already stated regarding his judgment as to the advisability of bracing the direct load-carrying steel reinforcement, it was obvious that he could scarcely agree with Mr. Waite that such bracing was not of much consequence, although he readily

concurred in the opinion that with such members as the battened channels used in these tests the difference was not great. He concurred with Mr. Waite in the opinion that the side or lateral pressure of the concrete against the encasing steel of this type of reinforcement had been very small in the tests made. Mr. Burr.

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16 and 23 January, 1912.

W. CAWTHORNE UNWIN, B.Sc., LL.D., F.R.S., President, in the Chair.

The discussion upon the Papers "Reinforced-Concrete Wharves and Warehouses at Lower Pootung, Shanghai," by Mr. S. H. Ellis; "The Direct Experimental Determination of the Stresses in the Steel and in the Concrete of Reinforced-Concrete Columns," by Mr. W. C. Popplewell, M.Sc.; and "Composite Columns of Concrete and Steel," by Mr. W. H. Burr, occupied these evenings.

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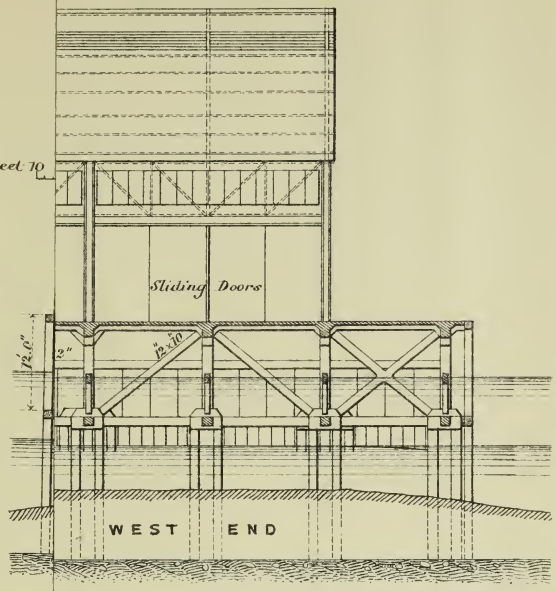
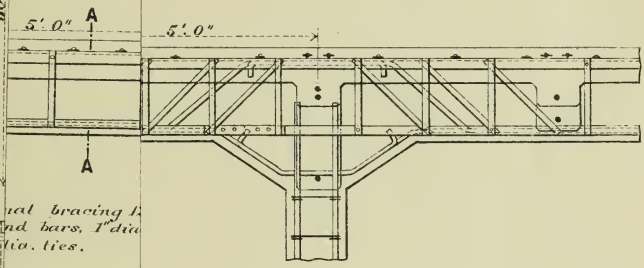
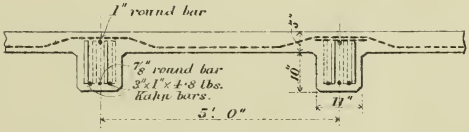
PLATE I.
REINFORCED-CONCRETE WHARVES
AND WAREHOUSES, SHANGHAI.

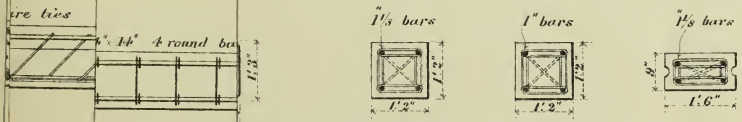


HARF.





CROSS SECTION OF BEAMS B B (Fig. 4)



CROSS SECTIONS OF
LUMN. FACE PILE. INNER PILE. SHEET PILE.

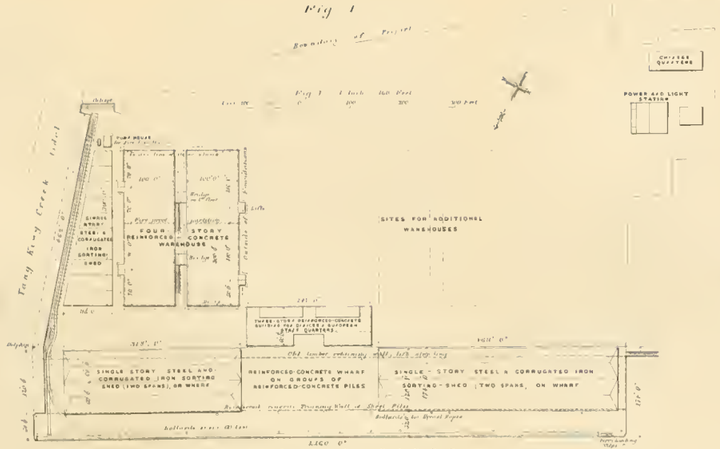
BEAM

AILS

Proceedi

S. H. ELLIS.

REINFORCED-CONCRETE WHARVES AND WAREHOUSES, SHANGHAI.



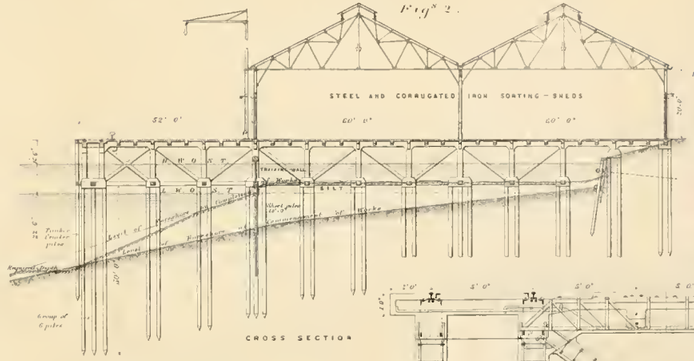
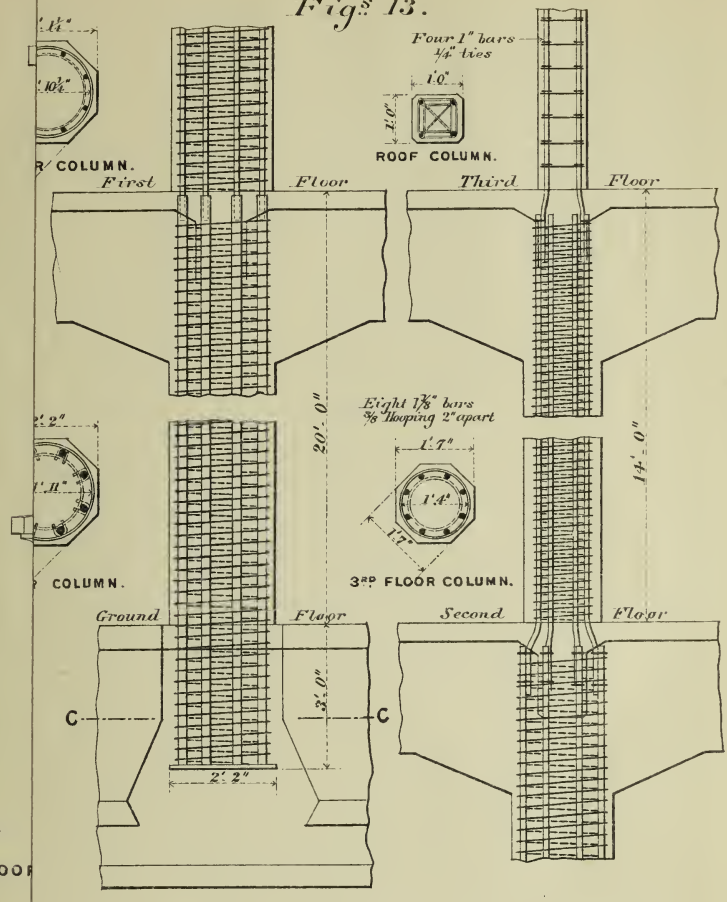


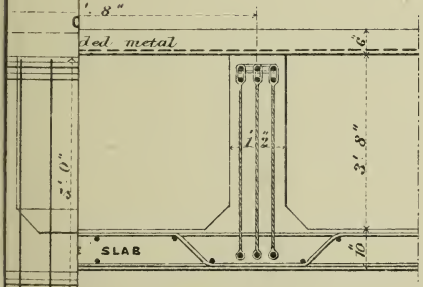
Fig 2.
STEEL AND CORRUGATED IRON SORTING-SHEDS
52' 0"
60' 0"
60' 0"
Transom Wall
Rail
Rail Girder
Rail Girder
Fender Timber
Fender Timber
Fender Timber
Level of Quay
Level of Godown
Timber Fender joint
Reinforced Concrete joint
Timber Fender joint
Reinforced Concrete joint
Timber Fender joint
Timber Fender joint
Timber Fender

PLATE 2.
REINFORCED-CONCRETE WHARVES
AND WAREHOUSES, SHANGHAI.

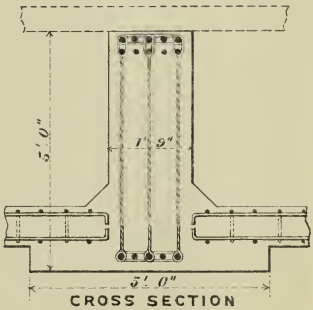
Figs 13.



WAREHOUSE COLUMNS.



LONGITUDINAL SECTION OF BEAMS.



CROSS SECTION
OF GIRDER AT A A.

S.H. ELLIS.

REINFORCED-CONCRETE WHARVES AND WAREHOUSES, SHANGHAI.

PLATE 2

SCALES

Figs 7 and 8. 1 Inch = 25 Feet

Figs 9, 10, 11, 12, 13, 13a & 14. 1 Inch = 5 Feet

Fig 7.

PART ELEVATION

STEEL AND CORRUGATED IRON SORTING-SHED.

CEMENT CONCRETE FLOOR (not reinforced)

LIME CONCRETE

Anchor-tie 20' 0" apart

CROSS SECTION.

Fig 8.

Brick-panel Walling

LIFT WELL

REINFORCED-CONCRETE FLOOR

DRY EARTH FILLING

LIME CONCRETE

PART CROSS SECTION

MOTOR HOUSE

LIFT WELL

PART LONGITUDINAL SECTION

BEAM

PLAN OF FOUNDATIONS.

WAREHOUSE

GIRDER

BEAM

SIDE OF WAREHOUSE

PLAN OF FIRST FLOOR

Fig 9.

Face Mark

Rubble Stone Pitching

Sheet piles

Bearing piles 12' 0" c. to c.

CROSS SECTION QUAY WALL REINFORCEMENT

PART PLAN

QUAY WALL

COUNTERFORTS 1 0" apart

Fig 11.

OUTER SPAN

INNER SPAN

LONGITUDINAL SECTION.

Expanded metal

CROSS SECTION WAREHOUSE FLOOR-BEAM.

Fig 12.

OUTER SPAN.

INNER SPAN

LONGITUDINAL SECTION.

CROSS SECTION WAREHOUSE FLOOR-GIRDER.

Fig 13.

2nd FLOOR COLUMN

ROOF COLUMN

1st FLOOR COLUMN

3rd FLOOR COLUMN

WAREHOUSE COLUMNS.

Fig 13a

PLAN CC.

Fig 14

TEST COLUMN

Fig 10.

EXTERIOR COLUMN

OUTER SPAN

INTERIOR COLUMN

INNER SPAN

LONGITUDINAL SECTION WAREHOUSE FOUNDATION - GIRDER.

CROSS SECTION OF BEAMS.

CROSS SECTION OF GIRDER AT AA.

SHEET

3 0112 098503722